MAR 13 1914

Engineering News

VOLUME 71 MARCH 12, 1914 NUMBER 11

New 15-Cu. Yd. Dipper Dredges for the Panama Canal

The new Panama Canal dredges "Gamboa" and "Paraiso" are nearing completion at the yards of the Staten Island Shipbuilding Co., in New York harbor. These dredges were ordered by the Isthmian Canal Commission as a part of the permanent equipment for the maintenance of the canal and for immediate use in completing the channel through Gatun Lake and the Culebra cut.

FIG. 1. GENERAL VIEW OF THE NEW PANAMA DREDGE "GAMBOA," AT THE YARDS OF THE STATEN ISLAND SHIPBUILDING CO., NEW YORK HARBOR

Each dredge has a displacement of about 1500 tons and is equipped with a 15-cu.yd. dipper, with extra 10-cu.yd. dippers for use in hard material. While the output of any dredge is hard to estimate until all conditions are known, it is expected that each dredge will be able to excavate 300,000 cu.yd. per month. Although the designs for these dredges follow very closely those for the Bucyrus dredge "Toledo," owned by Geo. H. Breymann & Bro., which has been at work for the past few years in Boston harbor, the new dredges embody many distinct advances in high-powered dipper-dredge construction.

DIPPERS—The 15-cu.yd. dippers have heavy manganese-steel lips and dipper-door hinges of forged steel 6x10 in. in cross-section. They measure 10 ft. 9½ in. in height from the bottom band to the upper edge of the lip. Fig. 2 affords a graphic idea of their size. This photograph shows 34 men standing on a platform hung halfway down the dipper.

The rock dippers are of extra massive construction and are fitted with heavy manganese-steel teeth. The dippers are hung on a 3¼-in. single-part wire-rope hoisting cable in accordance with the latest practice. These cables are the largest ever manufactured for a dredge. The dredges are designed to dig to a depth of 50 ft.

DIPPER HANDLE—The dipper handle is 72 ft. in length (see Fig. 3). The top bars and bottom bars are 2x12 in. in cross-section. The side plating is 1 in. thick. Long-leaf yellow pine is used in the construction of the sticks and white oak for the deadwood. The unusually long handle-end castings are held in place by a large number of 2½-in. horizontal and vertical through bolts. The weight of the dipper handle is 81,000 lb.

DIPPER BOOM—The boom which is shown in Fig. 4 is 62 ft. long, of the plate-girder type. Complete with the machinery it weighs about 113,000 lb. It is equipped with a steam-operated boom brake. A steam dipper-trip cylinder is mounted above the foot of the boom, and is connected to the latch bar on the dipper by means of an endless wire rope circuit, beginning at the upper end of the dipper handle, leading around the sheave on the stand just below the shipper-shaft, and thence around the sheave attached to the crosshead of the steam-dumping cylinder and ending at the dipper latch. It will be seen that this enables the dumping cylinder to act in any position of the dipper handle. The sheaves on the boom are heavier and larger than any ever before used for this purpose. The pins are 11 in. in diameter.

Saddle Block—The saddle block, another interesting feature in the design, is shown in Fig. 5. The purpose of this construction is to separate the slide plate into two parts, leaving a passage in the middle in order to permit the main hoisting rope to run in a straight line from the foot-of-the-boom sheave to the point-of-the-boom sheave. This obviates the necessity of a so called hump sheave, which is usually located near the upper end of the boom and is necessary on other dredges to lift the single-hoist rope clear of the saddle block.

The heavy unit construction has proved to be very valuable as it permits the guiding and the holding of the dipper handle in a much more secure manner, and also improves the meshing of the shipper-shaft pinions with the racks on the dipper handles.

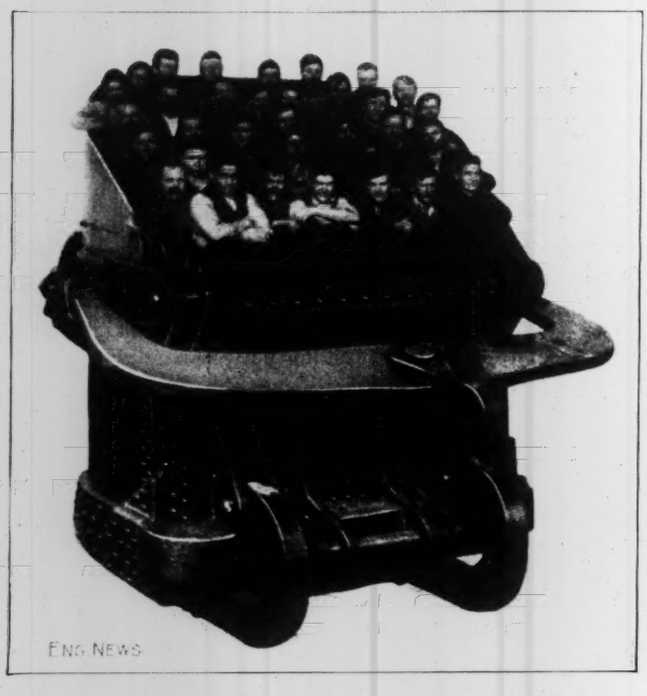


Fig. 2. Group of 31 Men in a 15-Cu.Yd. Dredge Bucket

Operating Machinery—The main hoisting machinery consists of horizontal, twin tandem, compound, condensing engines having cylinders 16 and 28 in. in diameter by 24-in. stroke. The valves, as in the common practice, are operated by the Stephenson link motion and are of the piston type on the high-pressure cylinders and of the flat-slide, double-ported, balanced type on the low-pressure cylinders.

As Fig. 6 shows, the engines are compound geared to heavy spur gears about 12 ft. in diameter. These gears are mounted on either end of a 16-in. main hoisting shaft which carries the drum. The drum is cast steel, of the differential type and is grooved to carry a 3¼-in. wire rope. This type of drum is characteristic of all high-powered Bucyrus dredges.

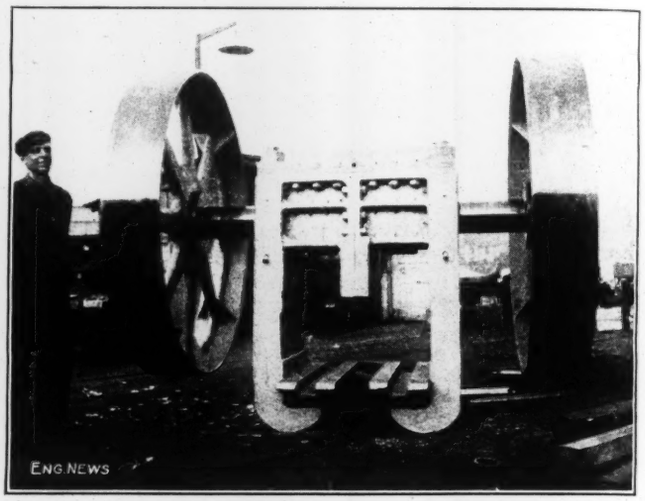


Fig. 5. Saddle Block and Brake Wheels for Panama Canal Dredges

The shape permits the maximum digging force and the slowest speed at the time when the dipper is digging and the angle between the hoisting rope and the dipper handle is the sharpest. As the dipper is hoisted the rope winds on to the larger diameter of the drum, which, of course, increases the speed when this is most desirable and when

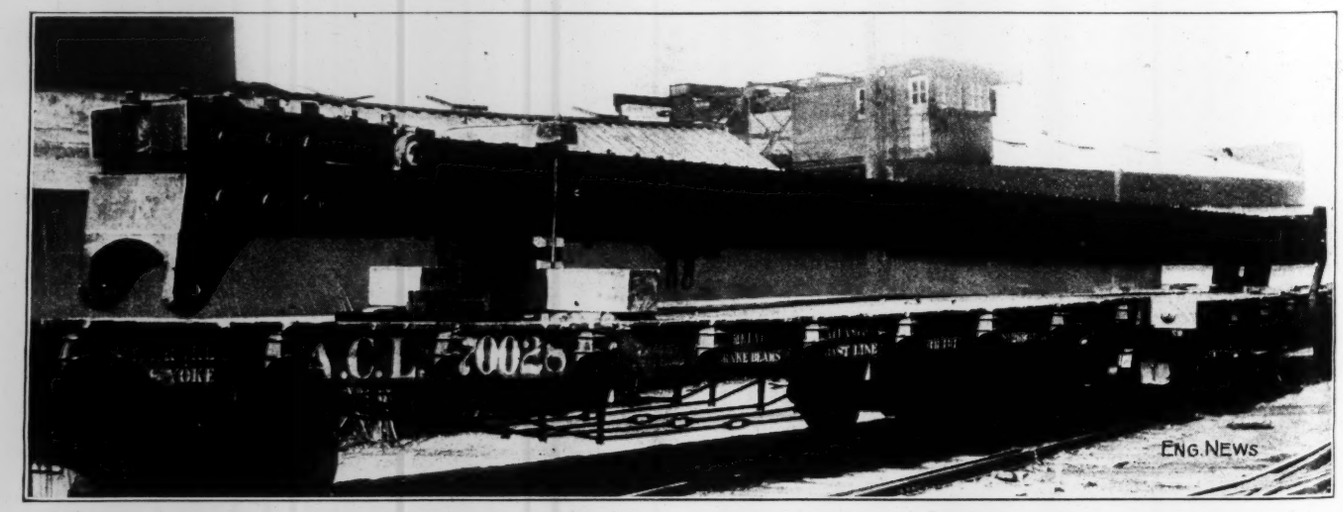


Fig. 3. Dipper Handles for 15-Cu.Yd. Dippers, Panama Canal Dredges, "Gamboa" and "Paraiso"

maximum bail pull is not required. With a view of lengthening the life of the rope the small diameter of drum on the new dredges has been increased.

The drum is, of course, mounted loose upon the forged-steel hoisting shaft above mentioned, and the power is applied through two wood-lined band frictions, which are mounted one on each side of the drum. These are operated by steam thrust cylinders. The reverse gear on the main engine is also steam operated.

An independent 12x16-in. double engine of the link-reverse type drives the swinging machinery. The swinging drum is cylindrical in shape and of cast steel, carrying two ropes which lead to the swinging circle. The latter is similar to the design on the "Toledo," being 24 ft. in diameter, heavily constructed of structural steel. The independent 12x16-in. double-cylinder backing engine is geared through friction clutches to a cylindrical-backing drum.

SPUDS—The forward spuds are of exceptionally heavy construction. They are 48 in. square and 72 ft. long, constructed with cast-steel diaphragms 5 ft. apart, and a continuous 10-in. I-beam in the middle of every side plate throughout its length. As shown in Fig. 7, the cast-steel spud point is unusually large. The collapsible spud foot is mounted on a 12-in. pin. The collapsible feature is of great advantage in permitting the spuds to break loose readily from a stick bottom.

SPUD OPERATION—The spud-operating machinery is a complete departure from other designs. On the largest preceding dipper dredges including the "Toledo," the spuds are driven through 2-part hoists and the drums operated by jaw clutches. On the new dredges the spuds are operated by 4-part hoists. This design has been adopted because of the enormous spud reaction which is occasioned when digging over the side of the dredge. The maximum reaction in this case on either of the spuds is in the neighborhood of 700,000 lb., which cannot be taken on two parts of rope of any available size.

It will be noticed in Fig. 1 that the spuds are placed farther back from the corners of the dredge than in previous designs. This is to make possible a swing of 180° instead of 120°, which has previously been the practice. The method of reeving the 2½-in. spud ropes may be seen from the general view of the dredge, Fig. 1.

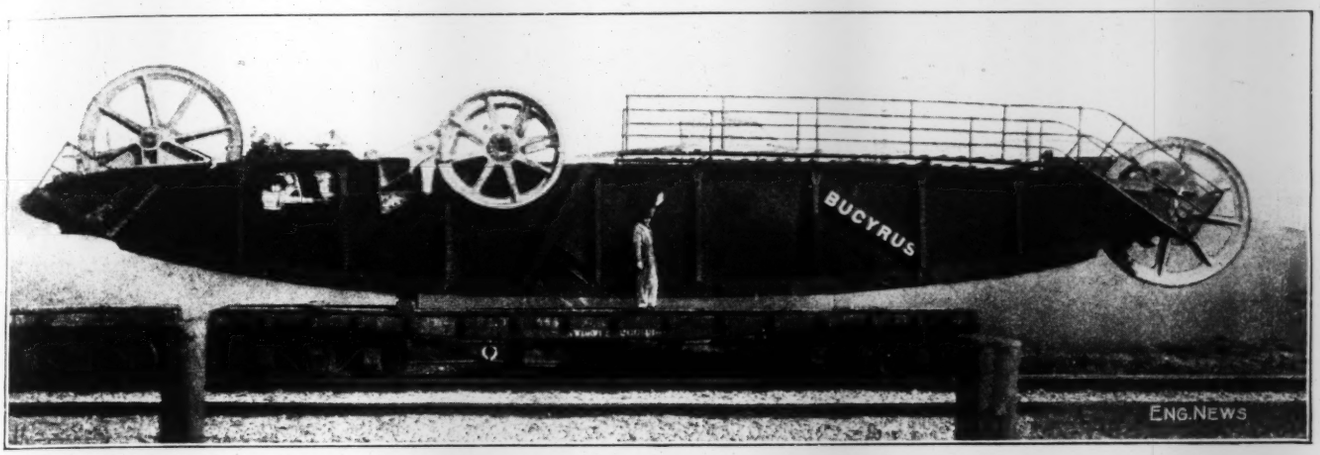


FIG. 4. DIPPER BOOM FOR PANAMA CANAL DREDGES; WEIGHT, 113,000 LB.

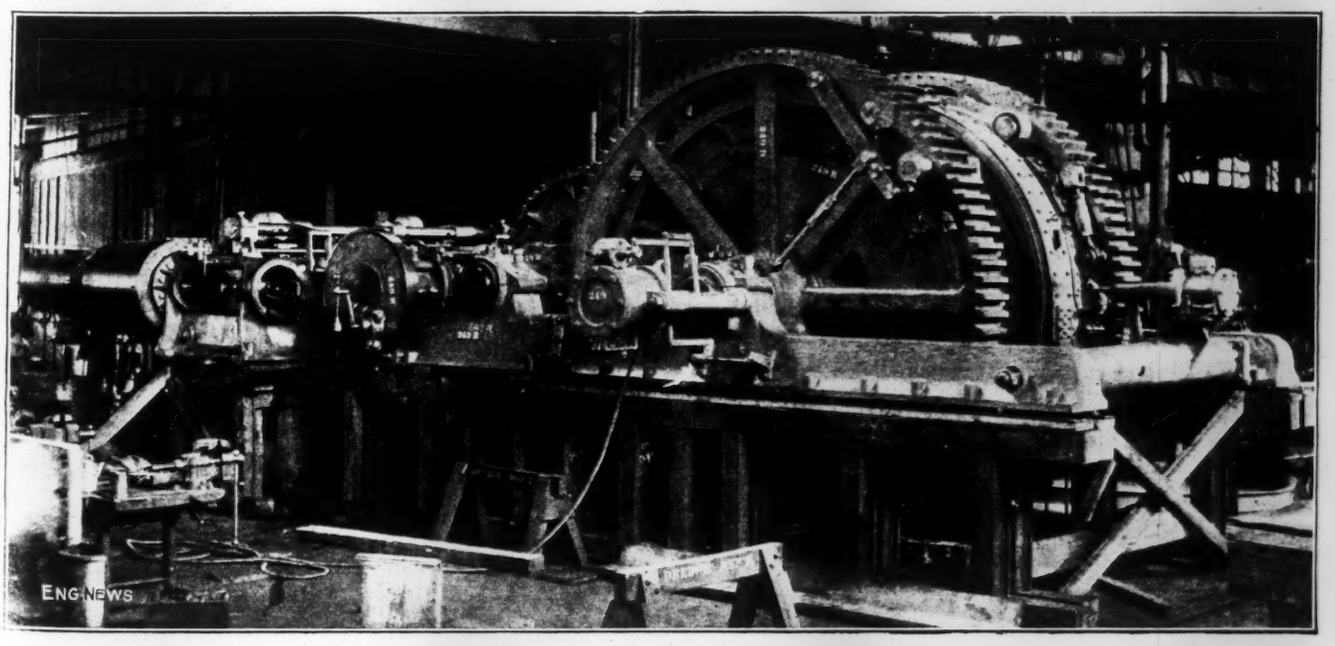


FIG. 6. MAIN HOISTING ENGINES FOR PANAMA CANAL DREDGES, "GAMBOA" AND "PARAISO"

The upper spud gate is a large hollow steel casting in the form of a padlock. It carries between the spud hinge pins the return sheave for the upper or pinning-up rope. On the lower part of this spud gate is hung a padlock

FIG. 7. COLLAPSIBLE SPUD FOOT

containing the return sheave for the lower or spud-hoisting rope.

The view of the spud machinery shown in Fig. 8 shows the throttle valve and the rotary-control valve for the steam-reversing cylinder mounted above the engine cylinders. It will be noted that they are connected to the same reach rod end. A single operating lever connection shuts off the steam in mid-position and operates the steam reverse gear to either the forward or the backing position, as the hand lever is moved one way or the other. A similar simultaneous control is applied in the case of the swinging engine, already described.

The stern spud machinery has also been considerably changed from other designs. Instead of operating this spud, which is of the trailing type, by means of a wire rope attached to the foot, as has been the common, though not the invariable practice in the past, the spud is hoisted and lowered by a rack and pinion drive.

The wire-rope method has been very successful when the digging is 30 ft. or less, but to greater depths there is always the danger of the operator allowing the spud to fall too fast, which is apt to cause the fastenings to be torn out when it is attempted to break the fall. The delay in making this repair is frequently long and often very costly as it is nearly always necessary to employ a diver to do the work.

HULL AND ITS EQUIPMENT—One of the unusual features of the design is the steel hull, the heavy construction of which is required to support the unusually heavy machinery within. It is distinctive particularly for the system of steel stiffening trusses and for the very elaborate equipment of the deck fittings, scow-moving winches, etc., and for the water-tight bulkheads, water-tight spud sponsons and other features not called for in a dipper dredge built for ordinary purposes.

An unusually complete steam plant is furnished. This consists of two Scotch marine boilers, each supplied with two Morrison suspension furnaces, a condenser with an air pump and a centrifugal circulating pump driven by an independent engine is included, as well as a closed

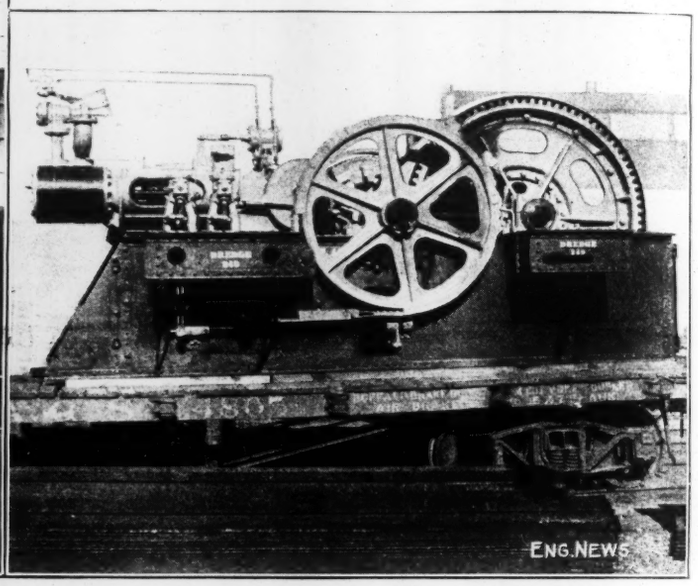


FIG. 8. SPUD-OPERATING ENGINES AND HOIST, SHOWING THROTTLE VALVE AND ROTARY CONTROL VALVE

feed-water heater, feed pumps, two fresh water tanks and a complete electric lighting plant. As the dredges are designed to burn either coal or oil they are equipped with two large oil-storage tanks, as well as coal bunkers.

Two 3-drum winches are supplied for moving scows. These are placed on the main deck, one on each side of the house, as may be seen from the general view of the dredge. They are operated by 6x6-in. horizontal engines placed inside the house.

The dredges are being tested in New York harbor, after which the booms and dipper handle will be taken down and the spuds removed. The dipper handles and spuds will be stowed on the decks of the dredges and the booms and dippers shipped separately. The first dredge will be towed to Colon, starting the latter part of February, and on its arrival will immediately be re-erected and placed in service. The second dredge will follow a few weeks later. The dredges were designed and are being constructed by the Bucyrus Co., of South Milwaukee, Wis. The erection work at Staten Island is in charge of Thos. Jardin, Superintendent.

A Railway Bridge Two Miles Long to link the island of Rügen in the Baltic Sea with the German mainland is being considered. The success of a 50-mile train ferry between Sassnitz, a northern point of Rügen, and Trelleborg, in Sweden, together with the growing popularity of the island as a holiday resort, has resulted in a passenger traffic that almost reached 100,000 in 1912.—"The Engineer."

Internal Stresses; with Application to Reinforced-Concrete Beams

A Series of Three Related Papers, with Diagrams

By Charles A. Gilchrist*

I. Stress at a Point

SYNOPSIS—A method of representing graphically the condition of stress at a point, doing for normal and shearing stress-components what the "ellipse of stress" does for resultant stress. General discussion of the variation of stress at a point.

It is customary to consider the condition of stress at a point from two distinct though harmonious viewpoints. One might be called the method of *resultant* stress, the other the method of *component* stress.

The condition of stress at a given point within a body is completely determined when we know the resultant intensity and the direction of stress on a given plane. On the other hand, we may substitute for these the components of the stress along the plane and perpendicular to the plane. In either case two things must be known: in the one case an intensity and an angle, and the other case two intensities.

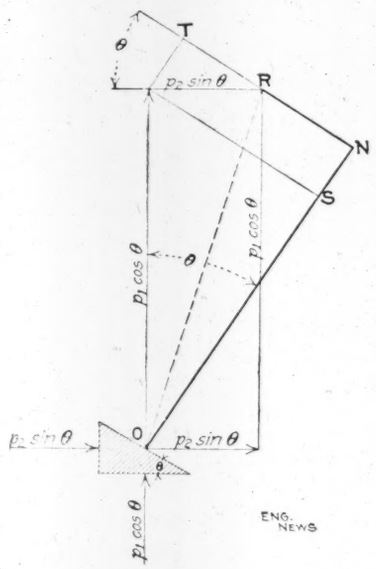


Fig. 1. Relation between Stresses at Any Point in a Stressed Body

Considering only stresses parallel to a given plane (such as occur in a simple beam), imagine a plane normal to this one to make one complete revolution about a point. For every position of the normal plane imagine a radius vector whose length and direction represent the intensity and direction of the resultant stress on the plane. It can be shown that the end of the radius vector will trace an ellipse, for the most general condition of stress that can exist at the point. But it must be observed that the ellipse alone does not completely represent the stress condition, for it is always necessary to know the angle between the normal to the plane and the radius, i.e., the obliquity of the stress. This obliquity is the eccentric angle of the ellipse. The semi-axes of the ellipse represent the *principal stresses* which have an obliquity (and shearing component) of zero. These semiaxes are axes of symmetry, upon which the resultant stress is the same as the normal stress.

The *ellipse of stress* is the graph for the method of resultant stress. It is the purpose to present herewith the graphs for the method of component stress, in the belief that they have not been presented heretofore.

Given two principal stresses p_1 and p_2, then the normal stress p and shearing stress q on a plane making an angle θ with p_2 are given by the equations:

$$p = p_1 \cos^2\theta + p_2 \sin^2\theta \qquad (1)$$

$$q = (p_1 - p_2) \cos\theta \sin\theta \qquad (2)$$

Inasmuch as some treatises covering structural mechanics contain no chapter on internal stresses, the derivation of these equations will be briefly given.

Fig. 1 shows an infinitesimal wedge with its perpendicular sides parallel to the principal stresses p_1 and p_2. The hypothenuse (assumed to be the unit of length) makes an angle θ with the direction of p_2, and the stresses upon it must be such that the wedge is in equilibrium. Bearing in mind that p_1 and p_2 are *intensities*, then the total principal stresses are $p_1 \cos\theta$ and $p_2 \sin\theta$, which are held in equilibrium by the total resultant stress OR, or by the normal stress ON in conjunction with the shear RN. Since the hypotenuse is unit length, total stresses and unit stresses upon it are the same, hence we have directly from the figure:

$$ON = p = OS + SN = p_1 \cos^2\theta + p_2 \sin^2\theta$$

$$RN = q = TN - TR = (p_1 - p_2) \sin\theta \cos\theta$$

which was to be shown.

That the locus of R is an ellipse apears as follows: The x and y coördinates of this locus in terms of the third variable θ, are:

$$x = p_2 \sin\theta, \text{ or } \frac{x}{p_2} = \sin\theta$$

$$y = p_1 \cos\theta, \text{ or } \frac{y}{p_1} = \cos\theta$$

The easiest way to eliminate θ is to square each side of each equation and add, whence:

$$\left(\frac{x}{p_2}\right)^2 + \left(\frac{y}{p_1}\right)^2 = 1$$

the equation of an ellipse with semi-axes p_2 and p_1.

The resultant stress p may represent a variable radius vector which is a function of the independent variable θ, while p_1 and p_2 are constants. We can now plot the curves or graphs of Eq. (1) and (2) in polar coördinates. To do this, assume numerical values for the constants, say $p_1 = 1$ and $p_2 = \frac{1}{3}$.

The heavy curve in Fig. 2, Case II, is the plot for normal stress. (Notice that any radius gives the normal stress in *that direction*, that is, on a plane which is always at right angles thereto.) Using the same constants in Eq. (2), we obtain the four-leaf clover shown dotted in Fig. 2, Case II. These two curves always go together, and jointly they give the stress for any direction of the plane.

Cases I to V, Fig. 2, show the chief forms these curves may assume. The most general cases are covered by Case II and Case IV. In Case II, the principal stresses are of like sign, while in Case IV they have unlike signs. Case III is the limiting case between these two, one principal stress being zero. Case I is the special case

*Designing Engineer, Bureau of Navigation, Manila, P. I.

where the principal stresses are equal and of like sign; and in Case V they are equal and of opposite sign.

The stresses at a point in a fluid are represented by Case I. Here the shear curve has dwindled down to nothing; the normal stresses are equal not only in the principal directions but in all directions.

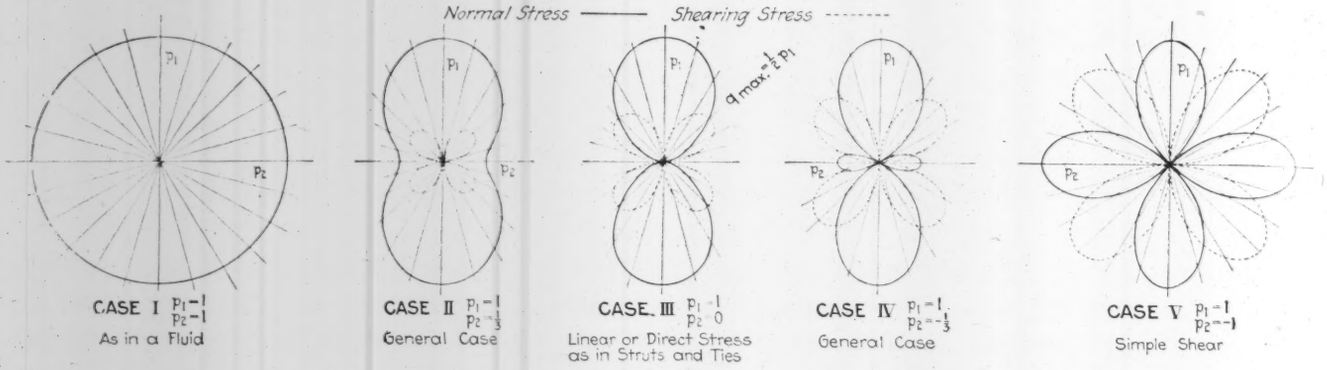

Normal Stress
Shearing Stress
p_1
p_2
$q_{max} = \frac{1}{2} p_1$
CASE I $p_1 = 1$ $p_2 = 1$
As in a Fluid
CASE II $p_1 = 1$ $p_2 = \frac{1}{3}$
General Case
CASE III $p_1 = 1$ $p_2 = 0$
Linear or Direct Stress as in Struts and Ties
CASE IV $p_1 = 1$ $p_2 = -\frac{1}{3}$
General Case
CASE V $p_1 = 1$ $p_2 = -1$
Simple Shear

FIG. 2. CURVES OF NORMAL STRESS (FULL) AND SHEARING STRESS (DOTTED) AT ANY POINT IN A STRESSED BODY

(The radius vector gives direction and intensity of normal or shearing stress-component on a plane at right angles to the radius vector.)

Case II is typified by the stresses at a point in an earth fill, where the horizontal and vertical stresses are both compressive but not equal. This is also the case in boiler shells, where we have both circumferential and longitudinal tensile stresses.

Case III, known as *linear* or *direct* stress, occurs in columns, cables, struts and ties. The diagram shows at a glance the entire absence of either shear or normal stress on a plane parallel to the axis of the piece. It also shows the shear on the diagonal plane, equal to one-half the direct stress and combined with an equal normal stress. This is the shear which in the diagonal member of a Pratt or Howe truss accounts for the entire vertical shear on the truss as a whole. Case II is simple stress, the other cases being compound stress.

As we progress from case to case, the difference between the principal stresses becoming greater and greater, it will be noticed that the shear figures grow in size and that they are always geometrically similar, being formed of four equal lobes. They are simpler and more symmetrical than the normal stress curves, and show at a glance the well known fact that the shears on any two planes at right angles are equal numerically.

Up to Case III, there has been no case where shear acts alone. Beyond Case III there are always two planes upon which *shear acts alone*, i.e., without normal stress, and this condition reaches its extreme in Case V, where this *uncombined* shear may reach an intensity equal to the principal stresses themselves.

The best example for Case IV is probably that of simple flexure where the principal stresses are always of opposite sign.

In Case V, the normal and shear curves are alike, with axes at 45° to each other. The maximum shear is *simple* shear since it occurs on planes where there is no normal stress. This is the condition along the neutral axis of a beam and on a cross plane at a point of contraflexure. The condition is also approximated on vertical surfaces under the edges of a punch, and it is ideally developed in a shaft under torsion.

Fig. 3 is a comparison of the curve for normal stress with the ellipse or curve for resultant stress. To avoid complicating the figure the shear curve has been omitted. Only one quadrant (between principal stresses) is shown. The dotted portion of the ellipse, although on the left, nevertheless represents the resultant stress for a plane whose normal is in the right quadrant, this being the case when p_2 is negative, in which case the resultant stress and the normal, starting together on p_1, rotate in opposite directions.

In Fig. 3, dotted lines are conditions for unlike signs, and solid lines conditions for like signs of the principal stresses. In this respect a change of sign does not change the figure of the ellipse, the ellipse of stress being the same for Case II and Case IV. Not so with the graph for normal stress, which gives very different figures if the sign of one principal stress is changed. In Case V (as in Case I), the ellipse of stress becomes a circle, but with normal and resultant stress-radii rotating in opposite directions. In Case III, the ellipse is a straight line and the resultant stress on any plane is parallel to p_1.

Many of the problems on the condition of stress at a point involve cumbersome equations. Here we have taken the simple case where the given stresses are the principal ones. In a beam under flexure the given stresses, although at right angles (being vertical and horizontal), are not the principal stresses, for there are shearing stresses in the same directions, and these are also known. In an earth fill with a sloping surface the known quantities are the intensity and obliquity of one stress (the vertical pressure on a plane parallel to the surface), and the maximum obliquity of any stress (the angle of repose or friction). To "orient" or determine the principal stresses from these data requires the formidable equation of Rankine for earth pressure. But what we want to emphasize here is that however complicated the problem may seem, the *condition of stress* cannot be more complicated than the general conditions we have discussed, and the difficulties are only incident to "orienting" the figure to its axes of symmetry.

The more general case when the given stresses, though not the principal ones, are at right angles, is usually solved before the special. This simply gives us equations for the identical graphs we have been discussing but with θ referred to axes oblique to the axes of symmetry. To determine the position and dimensions of these axes of symmetry (i.e., the principal stresses), the values of θ are found which make p a maximum and a minimum, respectively. But if we assume that Eq. (1) and (2) ex-

...at the most general condition referred to axes of symmetry, then the principal stresses may be found from ... given stresses by simply a simultaneous solution ... these equations.

If p_x and p_y represent two normal stresses at right angles, and q the shear accompanying each, we have:

$$p_x = p_1 \cos^2\theta + p_2 \sin^2\theta \qquad (1)$$

$$p_y = p_2 \cos^2\theta + p_1 \sin^2\theta \qquad (1a)$$

$$q = \tfrac{1}{2}(p_1 - p_2) \sin 2\theta \qquad (2)$$

equation (1a) being derived from Eq. (1) by putting $\theta + 90°$ for θ. The three quantities (p_x, p_y and q) on the left now become the given quantities, and the three quantities (p_1, p_2 and θ) involved on the right become the unknowns. The solution is now purely algebraic. By adding and subtracting Eq. (1) and (1a),

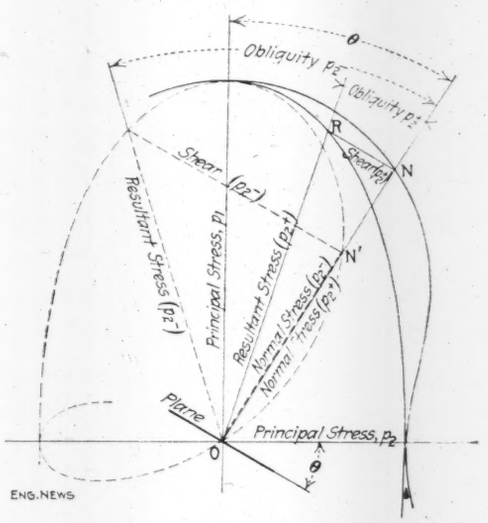


FIG. 3. COMPARISON OF ELLIPSE OF STRESS (FOR RESULTANT STRESS) WITH CURVE OF NORMAL STRESS

$$p_x + p_y = p_1 + p_2 \qquad (3)$$

$$p_x - p_y = (p_1 - p_2) \cos 2\theta \qquad (4)$$

By adding the square of (4) to the square of (2),

$$4q^2 + (p_x - p_y)^2 = (p_1 - p_2)^2$$

or,

$$p_1 - p_2 = \sqrt{4q^2 + (p_x - p_y)^2}$$

Adding and subtracting from (3) gives:

$$p_1 \text{ or } p_2 = \tfrac{1}{2}(p_x + p_y) \pm \tfrac{1}{2}\sqrt{4q^2 + (p_x - p_y)^2} \qquad (5)$$

which was to be derived.

To express θ in terms of p_x, p_y and q, divide (2) by (4):

$$\tan 2\theta = \frac{2q}{p_x - p_y} \qquad (6)$$

which was to be derived. Equations (5) and (6) are to be used in our second paper.

The law of the variation of stress at a point is a mathematical truth that, like all mathematical truths, knows no exception. It is true for all bodies whether or no they be elastic, homogeneous, isotropic or even solid. It simply asserts that *if* certain stresses are so and so, then other stresses must be so and so. If in any case the deduction is wrong, then the premise must be wrong.

A set of deductions similar to those that have been made for stress at a point can be made for *strain*, or intensity of deformation at a point. These deductions are also purely mathematical and have nothing to do with the physical properties of matter.

The deductions in regard to strain are quite independent of those in regard to stress but they show the same law of variation. The law for strain is Eq. (1) with strains substituted for the stresses p, p_1 and p_2. This similarity does not prove Hooke's Law, but it does show that *if* stress is proportional to strain in the two principal directions, then stress will be proportional to strain in every other direction.

The writer first felt the need of such considerations as are here discussed when studying the design of shear or web reinforcing for reinforced-concrete beams. At once the question arose: How can we hope to learn anything about these stresses which are not only compound but vary throughout the beam, unless we know all about the *point* first? Armed with the complete analysis of the condition of stress at a point we are prepared to consider problems of the distribution of stress from point to point, problems involving the elastic properties of the material. These problems cannot fairly be called simple and easy, and it is certain their solution cannot be accomplished without a knowledge of the point conditions first. Professor C. E. Greene says in the preface to his "Structural Mechanics" that time given to the study of internal stresses is well expended and the results shed much light on various problems.

A Useful Hint to Young Engineers—"The man for whom every employer of men is searching, everywhere and always, is the man who will accept the responsibility for the work he has to do—who will not lean at every point upon his superior for additional instructions, advice or encouragement.

"There is no more valuable subordinate than the man to whom you can give a piece of work and then forget about it, in the confident expectation that the next time it is brought to your attention it will come in the form of a report that the thing has been done. When this master quality is joined to executive power, loyalty and common sense, the result is a man whom you can trust. On the other hand, there is no greater nuisance to a man heavily burdened with the direction of affairs than the weak-backed assistant who is continually trying to get his chief to do his work for him, on the feeble plea that he thought the chief would like to decide this or that himself. The man to whom an executive is most grateful, the man whom he will work hardest and value most, is the man who accepts responsibility willingly."—From "The Training of a Forester," by Gifford Pinchot.

Polluted Water-Supplies and Mortality of Children under two years of age, particularly mortality from diarrhea and enteritis, was the subject of a paper by Dr. Allan J. McLaughlin, surgeon U. S. Public Health Service, Washington, D. C., read before the Indiana Sanitary and Water Supply Association late in February, 1914. The high infant mortality commonly occurring in summer seems to be independent of water-supplies in most cases, but there is a type of enteritis among children which may occur at any time in the year. This type "is associated with and probably dependent upon a sewage-polluted water-supply." It sometimes "shows a very high rate of prevalence in the winter and spring months." Thus Wyandotte, Mich., with a grossly polluted river water-supply, has high enteritis rates, with "an abnormal prevalence in the spring months." The Wyandotte enteritis rate is higher in March and April than the Benton Harbor rate (water from deep wells) for July and August. Marquette, Mich., with winter and spring pollution of its water-supply, has high enteritis rates at the same time, while Sault Ste. Marie, Mich., with water pollution chiefly during the boating season, has high enteritis rates in the summer and fall. Coming east to Cohoes, N. Y., there was a heavy drop in enteritis after a mechanical filtration plant was put in operation in July, 1911. Thus, the January-June enteritis rate per 100,000 in children under two years old at Cohoes during the past five years was: 1909, 144; 1910, 192; 1911 (filter plant in operation in July), 192; 1912, 80; 1913, 48.

English and Metric Scales on Plans and Some New Conversion Tables

(With Insert Sheet of Tables)

By H. P. Quick*

SYNOPSIS—The article describes the present confusion which exists where American engineers have to prepare plans for foreign engineers. The author would express all scales on drawings as proportions for ease in converting from English to metric systems or vice versa. Three new conversion tables and a new scale relation table are reproduced on an insert sheet in the form found most useful.

In connection with engineering work in foreign countries where practically all plans are made to the metric system, when American engineers are called on to furnish plans, there frequently arises a troublesome question: What scales shall be used for the plans to be sent to resident engineers or to foreign manufacturers? Both use the metric system, but the American manufacturers and draftsmen use the English system. Shall the plans be made in the $\frac{1}{8}$-in., $\frac{1}{10}$-in. or centimeter system, and shall they be dimensioned in feet and inches, in feet and tenths, meters and decimals, or millimeters, or a combination of English and metric units.

In practice, it has been found desirable to make those plans which are to be compared and used in the field with foreign plans, in the metric system direct, or to draw them to an English scale, approximating or equal to metric, and thoroughly to dimension them in the metric system. Then they may be used directly in the field, or they can be traced by the local staff, for filing with the government if necessary, without redrawing.

Plans have also been made and dimensioned in the English system, but have had the scale marked as the nearest metric proportion. Then those familiar with the metric system only know they may be scaled with the metric scales for approximate measurements at least. We have also occasionally put both English and metric values of measurements on the plans drawn to either scale.

Owing to this confusion one can see that it would remove nearly all trouble if engineers would adopt the metric system to the exclusion of others. As this cannot be done, it would greatly clarify matters if at least the decimal or one-tenth system were substituted for the inches and eighths, and only those scales used which correspond in proportion to those used in the metric system, like the following:

1:10	1:100	1:500
1:20	1:200	1:800
1:40	1:250	1:1000
1:50	1:300	1:2000
1:80	1:400	1:10000

However, it is not likely that even this will come to pass very soon. There is a great deal of confusion about scales, even among technical men, as writers on this subject do not present it properly and there is absolutely nothing about it in most engineering books. The following is presented, therefore, as a statement of the relation between English and metric scales, in the hope that it will simplify the matter in some minds and perhaps lead to its presentation in proper form in handbooks and textbooks.

All scales of plans are intended to show the proportion or relation between the plan and the actual work or thing shown and really should be stated in the form of a proportion as all scales can be reduced to, or are converted or derived from, proportions. For instance, a scale may be written "3 in. = 1 ft."; this is the same as saying "¼ in. to 1 in." or "¼th size" or "1 in. = $\frac{1}{3}$ ft." or "1 in. = 4 in." These all mean the same thing, namely, that 3 in. on the plan = 1 ft. or 12 in. in distance, or length or dimensions of the thing shown. They can thus all be represented fundamentally by the simple proportion of 3:12 or 1:4. Or take the metric scale of "25 cm. = 1 m." (100 cm.). This is the same as "1 cm. = $\frac{1}{25}$ m." (4 cm.), or "250 mm. to 1 m." (1000 mm.), or "25 dm. to 1 m." (10 dm.) ; all are reducible to "1 cm. = 4 cm." or "1 mm. = 4 mm." or 1:4. This is not, as many suppose, one of a certain measure to four of some other measure not mentioned, but one of anything to four of the same thing, and may be taken as inches, feet, yards, miles, centimeters, meters, pencils, paper, tape, or any convenient form of measure, and in fact, one could use any one of these units or things for any plan of that scale if marked by subdivisions, whether they corresponded to the established unit of measurement or not, for they could later be compared if necessary.

Unfortunately, one will find the English scales, which can be bought, not marked with the value proportionally stated; instead, whole numbers or fractions are used to designate the scales, that is, the relation of the plan to the distance, thus, $\frac{3}{32}$ (inches on the plan = 1 ft. in distance) and 20 (feet in distance = 1 in. on the plan). The signs, words or figures given in parentheses here are left on the scales to be filled in mentally. Below is a complete list of such scales as are in general use with the deficiency in meaning supplied in parentheses.

ARCHITECTS' SCALES

$\frac{3}{32}$ (in. = 1 ft.)		¼ (in. = 1 ft.)	
⅛ (in. = 1 ft.)	(8 ft. to 1 in.)	1 (in. = 1 ft.)	(1 ft. to 1 in.)
$\frac{3}{16}$ (in. = 1 ft.)		1½ (in. = 1 ft.)	
¼ (in. = 1 ft.)	(4 ft. to 1 in.)	2 (in. = 1 ft.)	
⅜ (in. = 1 ft.)		3 (in. = 1 ft.)	
½ (in. = 1 ft.)	(2 ft. to 1 in.)	4 (in. = 1 ft.)	
		6 (in. = 1 ft.)	

ENGINEERS' SCALES

5 (ft. to 1 in.)	50 (ft. to 1 in.)
10 (ft. to 1 in.)	60 (ft. to 1 in.)
20 (ft. to 1 in.)	80 (ft. to 1 in.)
30 (ft. to 1 in.)	100 (ft. to 1 in.)
40 (ft. to 1 in.)	

These are all reducible to, or derived from, proportions as shown by the appended table explained further on.

This much being firmly fixed in one's mind, it is easy to understand the relation of, and to convert to and from, metric and English scales. As stated, metric scales are merely proportions or ratios, and are always so stated on plans and are generally multiples of 10, e.g., 1:20, 1:10, 1:50, 1:500, 1:1000, so that any scale so stated can be considered either from the English or metric standpoint, as happens to be convenient, and whatever scale is in hand can be used; either a 1-ft. rule, or a meter rule, or the finely graduated scales or tapes of either class.

*Pearson Engineering Corporation, 115 Broadway, New York City.

right here, however, one gets into trouble because metric scales themselves are frequently not marked by the system of proportions or by proportional signs, but are given as decimals or fractions. For the ratios 1:100 and below, one will find the standard metric scales marked decimally; thus, .01, .0125, .02, .025, .03, .05. Above this value, i.e., 1:100, they are marked as proportions, viz.: 1:200, 1:400, 1:500, 1:800, 1:1000, 1:2000.

As to the scales to be used in measuring any given plan, take a plan having the scale 1:50, that is, 1 in. to 50 in., or 1 cm. to 50 cm., depending upon the system in which one is thinking. If in English, one will mentally divide 50 in. by 12 in. and see that the result is near 4 ft., so the scale of 1 in. to 4 ft. may be used for approximation. If greater accuracy is required, he can use the decimal scale of 50, in which he will find 50 divisions to the inch and can read inches directly. If one is using a combination metric and English rule, he can read off the inches directly on one side and turn over to the corresponding centimeters or millimeters on the other, or *vice versa*. If thinking or working in the metric system alone, one should take the scales labeled in decimals, viz.: .02, .025, and so on, which may be found to be written $^{2}/_{100}$ and $^{25}/_{1000}$, or 2:100 and 25:1000, or $^{1}/_{50}$ and $^{1}/_{40}$, or 1:50 and 1:40. Of these, the 1:50 or 0.02 scale is the one to use if metric measurements are wanted in the above instance. As on this scale the figures given are from 1 to 15 meters divided into 10ths and 50ths, meters and 10ths may be read directly corresponding to the feet and inches read from the English scales.

So much for the scales to be used. Now as to reading the metric values, assuming the English to be well understood, it is well to remember that, just as with English scales it is convenient to think of, and read off, the distance in number of feet (and inches or tenths) or miles per inch or fractions of the inch on the plan, so in the metric system it is convenient to think of the number of meters (and decimals) or kilometers per centimeter or millimeter of the plan, the smallest unit always being that of the plan. Thus, it is usual and convenient for scales of 1:100 and upward immediately to take the result of mentally dividing both sides of the proportion by 100 (for 1 cm. = $^{1}/_{100}$ m.) and read the proportion in this way: 1 cm. = 1 m., 2 m., or 3 m., etc., as the case may be, up to the point where the meters become decimeters, which would be the scale of 1:10,000. Then it is usual to think in kilometers per decimeter and so on to the scale of 1:100,000 when the meters become kilometers and we think in kilometers per centimeter.

To show the relation of all forms of scales both English and metric to themselves and the fundamental proportion or ratio, the writer, some seven years ago, prepared a table, which has been in use privately since then and which is here given publicly for the first time. This shows all the scales in general use between 1:1 and 1:10,000,000, that is, full size and 1 in. = 160 miles, or 100 km. to the cm. A great many intervening values are given between the standards which are not used but they are inserted to show the progression and derivation and relative situation of those that are used. Only those proportions which are divisible by 5 or 10 are given except where the scale in common use requires it otherwise, like 1:384, which is the equivalent for $^{1}/_{32}$ in. = 1 ft.

In examining this table it will be found that every ratio given corresponds to some even or whole number or common scale, and this is the reason for the use of so many proportions and of the particular ones given. The intervening ones not representing any scales in common use would be of no special value. It would be impracticable to give all the map scales in use so only a few are given which have been found in use between the even 50,000's.

There are also given herewith two metric and English conversion tables never before published but used by the writer and others associated with him for the last six or seven years. These have been found very convenient, especially for close approximation, and correct within very small fractions of an inch—which is good enough for most engineering work. For instance, the table for converting meters and millimeters into feet and inches runs to 309.999 m., and by additions and multiplications, can be carried indefinitely to any dimension or distance. This table gives the values of conversion of meters into feet, inches and fractions, and feet and decimals carried to four places, directly under each other. It serves for the conversion of millimeters from 1 up to 999 into inches and decimals, or feet and decimals, or feet, inches and fractions. The other table for converting feet and inches into meters and millimeters is very convenient for almost any value, no matter how large or small except below 16ths. It reads directly up to 99 ft. 11 in., and all the fractions by 16ths. Of course, a few additions are necessary but the tables save a great deal of mental calculation and the writer believes are in a better form than any that have been published. A fourth table is a comparison of standard linear units for readily reading the value of one of any unit in terms of any of the other units.

The Improvement in the Status of the Engineer*

By C. H. Snyder†

The status of the Civil Engineer in this vicinity has much improved in the last ten years. Practically no work of importance, with engineering features, is undertaken now except under the advice of a trained engineer; and this fact has given us experience to undertake still larger and more important enterprises.

Ten years ago, in the building trade, structural designing was nearly all done by architects themselves, with such assistance as they could get from contractors in the various trades. Today, few important buildings are handled in this way, and I confidently look forward to the time when even the highway bridge business will be handled as it should be.

One cause for the more extensive employment of engineers in building work has no doubt been the improved character of the buildings themselves. Our standard of construction has risen, and we have both better designing and better execution of the design. It is the exception when steel work is not well executed. The quality of material is specified, the workmanship inspected and the erection superintended by more or less competent men. Just think! Ten years ago there was practically no inspection of shop work here. A perfunctory look by the architect was considered sufficient. We now make a serious, and, I believe, successful effort, to see that the owner gets what he is paying for.

Concrete work, too, is getting very careful consideration. Our knowledge of designing this important building material has advanced by leaps and bounds and our testing of the materials, and inspection of the mixing, workmanship, etc., have advanced very materially, albeit we have perhaps lagged a

*Extract from the Presidential address delivered before the San Francisco Association of Members of the American Society of Civil Engineers, Feb. 20.

†Designing and Constructing Engineer, 251 Kearney St., San Francisco.

little on mixing and installation. The violent controversies of structural association days have nearly passed, and all good engineers are willing to use the construction best adapted to the purpose to be accomplished. Few of us believe in building a campanile of reinforced concrete, or in putting the largest part of a steel frame underground.

Fireproofing, also, is being studied carefully in the light of experience gained in the two great fires of the last decade, thanks, in a large measure, to the excellent engineers in the employ of the insurance companies. The resulting classification and grading of rates has been a powerful incentive to the construction of buildings of proper type to minimize the chances of future fire catastrophies. The speculative builder, of course, is still with us, but our building codes are being improved by the efforts of our members and being much better enforced, also due to our members.

A striking feature of development in the last few years is the large number of engineers engaged in the execution of engineering structures. No general contractor engaged in handling work of magnitude would dream of carrying on his business without an engineer, and many of our large contracting companies have engineers for their executives, and good ones, too.

Within my own recollection steelwork was contracted for by men to whom the stresses in a roof truss were a profound mystery. The difficulties attending negotiations under these circumstances were sufficiently great to induce the contracting man to take along an engineer to discuss the technical points; and it was but a step to promoting the engineer to the sales end of the business. This change took place in the steel business afout 15 years ago, and has since spread to machinery and electrical devices, as well as to building materials.

Our code of ethics is to give our client, and the fellow members of our profession, a square deal. In other words, apply the golden rule. Not much more than that is required to guide us in our relations with our fellow men, notwithstanding the various rules defining our ethical code. Few cases, if any, arise in which a moment's consideration will not give us the proper line of conduct. No one is so well qualified to judge the actions of an engineer as another engineer, and the higher the standard of the Individual, the higher will be the ideals of the profession. To me there has been a decided improvment in engineering ethics in San Francisco.

Let us not lose heart, then, in fear that our profession is not making the progress that it should; that it is disgraced by a few unworthy members; not accorded credit commensurate with the abilities of its members; is ill paid, and of such standing that we should advise our sons to join the hod-carriers' union at $7 per day. I say again that ten years have shown a great improvement here in the status and attainments of the civil engineer. Let us hope that the next decade will show an equal or greater progress.

"Honor and shame from no condition rise
Act well your part, there all the honor lies."

Semi-Elliptical Sewer Sections

By John H. Gregory*

Of the various sections adopted for large sewers, especially those carrying sewage rather than storm water, among the best is the so called semi-elliptical section. This section has much to commend it, not only from the viewpoint of hydraulics but also from the viewpoint of construction. Further, the section is well adapted for carrying the superimposed load of backfilling.

Some years ago the writer had occasion to work out a set of standard sections for sewers of this type in connection with the preparation of plans for a large sewerage project. A diagram giving the discharging capacity of the sewers when running three-quarters full depth, for which condition of the flow the sewers were to be designed, was also made, as well as a diagram giving approximately the relative discharging capacities for all depths of flow. These sewer sections, together with the hydraulic diagrams, have been found very convenient by the writer, and by other engineers familiar with them, not only for purposes of design but also for estimating purposes, and they are presented herewith in order that they may possibly be of service to still other engineers who are called on to design or estimate on work of this nature.

The semi-elliptical section adopted, and which it is assumed would be built of concrete, is shown in Fig. 1. It should be stated that this section is better adapted for sewers 6 ft. and over in diameter than for sewers less than 6 ft. in diameter. From an examination of the section it will be noted that the several dimensions are given in terms of the diameter D. The functions of the diameter were so chosen that, starting with any diameter, D, in feet, and with increments of 3 in. in the diameter, the resulting dimensions will come out in whole inches or in inches and fractions of inches in common use, as, for example, quarters, eighths or sixteenths.

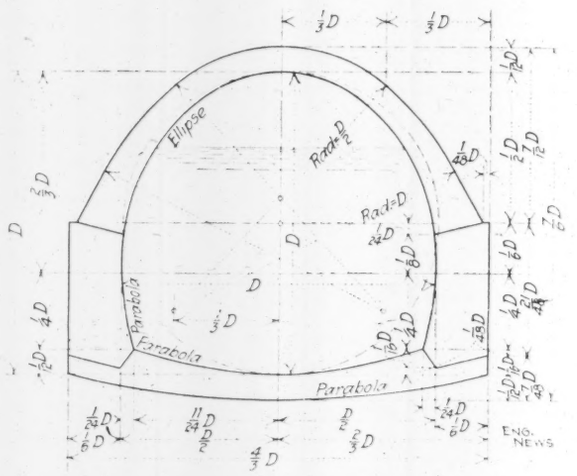


Fig. 1. Standard Section of Semi-Elliptical Sewer

The section shown is suitable for use only where the conditions are such that the side walls will be firmly supported by the sides of the trench. Where these conditions cannot be obtained the side-wall section can be readily modified to conform to the shape of the side wall commonly used in large aqueducts.

Without attempting to call attention to all of the inside dimensions it should be noted that the horizontal and vertical diameters of the section are the same; that the horizontal diameter is located at a depth below the inside crown of the arch of two-thirds the vertical diameter; and that that portion of the section above the horizontal diameter is a true semi-ellipse.

For computing the volume of masonry in the section the following formulas will be found of service:

Gross area on outside lines	$1.2651\ D^2$
Area of section inside	$0.8176\ D^2$
Net area of masonry	$0.4475\ D^2$

For convenience of reference the gross area on the outside lines, the area of the section inside and the net area of the masonry, as well as the volume of masonry in cubic yards per linear foot, for sewers varying in diameter by feet and half feet from 6 ft. to 13 ft. 6 in. in diameter, are given in the accompanying table.

The velocity in and corresponding discharge of semi-elliptical sewers of the section shown, when running three-quarters full depth, can be readily obtained from the diagram, Fig. 2. The diagram is based on Kutter's formula, with $n = 0.015$, and covers the range in di-

*Of Hering & Gregory, Consulting Engineers and Sanitary Experts, 170 Broadway, New York City.

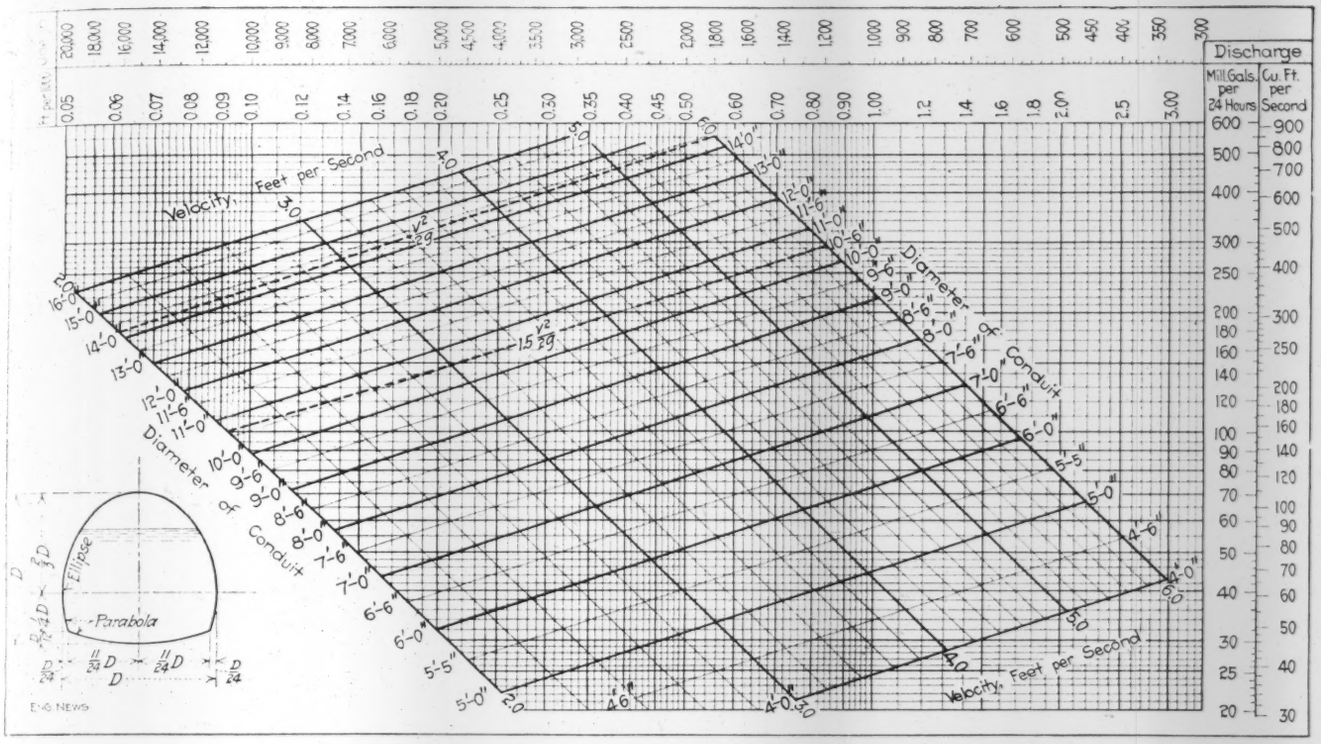


FIG. 2. DIAGRAM GIVING DISCHARGE OF SEMI-ELLIPTICAL SEWERS RUNNING THREE-FOURTHS FULL DEPTH
(For Kutter's formula with n = 0.015)

AREA OF AND VOLUME OF MASONRY IN SEMI-ELLIPTICAL SEWERS

Inside diameter, ft., D (1)	Gross area outside lines 1.265 D^2 (2)	Area in sq.ft. Area of section inside 0.8176 D^2 (3)	Net area of masonry 0.4475 D^2 (4)	Volume of masonry, cu.yd. per lin.ft. 0.01657 D^2 (5)
6	45.54	29.43	16.11	0.597
6½	53.45	34.54	18.91	0.700
7	61.99	40.06	21.93	0.812
7½	71.16	45.99	25.17	0.932
8	80.97	52.33	28.64	1.061
8½	91.40	59.07	32.33	1.197
9	102.5	66.23	36.25	1.342
9½	114.2	73.79	40.39	1.496
10	126.5	81.76	44.75	1.657
10½	139.5	90.14	49.34	1.827
11	153.1	98.93	54.15	2.005
11½	167.3	108.1	59.18	2.192
12	182.2	117.7	64.44	2.387
12½	197.7	127.7	69.92	2.590
13	213.8	138.1	75.63	2.801
13½	230.6	149.0	81.56	3.021

eters and velocities ordinarily met with in practice. The diagram is practically self-explanatory, but it may be said that from any point inside the diagonal lines the corresponding diameter, velocity, slope and discharge can be read.

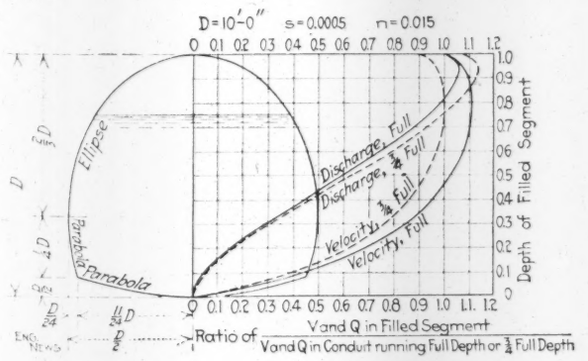

D=10'-0" s=0.0005 n=0.015
Ellipse
Parabola
Discharge, Full
Discharge, ¾ Full
Velocity, ¾ Full
Velocity, Full
Depth of Filled Segment
Ratio of V and Q in Filled Segment / V and Q in Conduit running Full Depth or ¾ Full Depth

FIG. 3. DIAGRAM GIVING RELATIVE VELOCITY AND DISCHARGE OF SEMI-ELLIPTICAL SEWERS RUNNING PARTLY FULL

It is often desirable to know the velocity head and the loss of head at entrance, or the sum of the two, and either or all of these quantities can be obtained from the diagram. Thus, to find the head required to produce a velocity of 3 ft. per sec., it is only necessary to find the intersection of the velocity line 3 with the dotted line marked $\frac{v^2}{2g}$ and read the velocity head corresponding thereto on the scale marked Slope in Feet per 1000, or 0.14 ft. The loss of head at entrance would be found by dividing the velocity head by 2, assuming that the loss of head at entrance would be $0.5\,\frac{v^2}{2g}$. The sum of the velocity head plus the loss of head at entrance is found in the same manner as the velocity head alone except that the dotted line marked $1.5\,\frac{v^2}{2g}$ is to be used in finding the intersection with the velocity line. For a velocity of 3 ft. per sec. the value of $1.5\,\frac{v^2}{2g}$ is seen to be 0.21 ft.

The approximate relative velocity and discharge for any depth of flow, referred to the sewer flowing full or three-quarters full depth, can be obtained from the diagram given in Fig. 3. By means of this diagram, together with the diagram given in Fig. 2, the velocity and corresponding discharge of the sewers when running either full or at any less depth of flow can be obtained.

In conclusion it should be pointed out that the dimensions given for the masonry sections are a *minimum* and that not only would the best of materials and workmanship be required but also careful inspection. Where these conditions cannot be obtained or where the sewers would be required to carry heavy loads the sections should be reinforced with steel or the dimensions increased, especially the arch and side-wall sections.

The Testing of Sand for Use in Concrete--II. Computing and Using Results

By Cloyd M. Chapman*

In Part I of this article published in Engineering News, Feb. 5, 1914, p. 306, there were described in detail the methods and manipulations employed in the laboratory of Westinghouse Church Kerr & Co., in routine tests of sands to be used on their concrete work. These tests are applied to any and all sands which are to be used on important work and from the data thus obtained determinations are made of the numerical value of the various characteristics of the sands and of mortar and concrete made from them. The methods used in computing these numerical values, in determining the relative quality of the sand and in specifying the proportions of cement and sand which shall be used for the particular work in hand are described in the present article. As each test is taken up, comment will be made on reasons for making the test, its general value as a reliable test, and its usefulness to the engineer in assisting in specifying how the sand shall be used on the job.

SAND TEST DATA SHEET * * Westinghouse Church Kerr & Co., 37 Wall St., New York

| | | |

"Contract Number" of the job from which the sample was received, the "Name of the Contract," the "Name of the Dealer" furnishing the sand, the location of the deposit or "Source" of the sample, the "Mark" on the sample, if any, such as the car number or name of barge on which the sand was received, and the "Laboratory Mark" assigned to the sample upon its receipt, all as given in spaces (1) to (7) respectively on the data sheet and placed in spaces (*a*) to (*g*), respectively, on the report-form.

Moisture contained in the sand as received is computed from the loss in weight of a 120-gram sample when dried, and the result is recorded on the report form at *h*.

$$\text{Formula:} \quad (h) = \frac{\mathbf{120} - (22)}{\mathbf{120}}$$

This test is of sufficient accuracy, as a test, but owing to the difficulty in securing representative and reliable samples, truly indicating the amount of moisture in a sand, the test is often not of very great value. With a carefully taken and properly sealed sample this test would show with sufficient accuracy the amount of moisture present. The value of the test is twofold. First: It shows, in those cases where sand is purchased by weight, or is shipped by weight, the amount of excess water which is being paid for as sand or the amount of freight charges that have to be borne on account of the water. Second: The test shows how much sand to weigh out in making other tests in which a definite net weight of dry sand is wanted without drying the sand; as, for instance, in the tensile strength test.

Per Cent. of Material Retained on 4-Mesh Sieve—This percentage is entered on the report blank in space (*i*)

$$\text{Formula:} \quad i = \frac{(10) - (11)}{(8) - (9)}$$

The purpose of this test is to determine the amount of material in the sample which is too coarse to be considered as sand in determining proportions for use on the job. The line of demarcation between sand and gravel or between fine aggregate and coarse aggregate for concrete has very generally been considered as lying near the 4-mesh size. For some purposes this is too coarse, while for other uses there is no objection to even coarser material. The 4-mesh or often the ¼-in. screen has found very common use as the dividing sieve for separating fine from coarse aggregates and as there is at present no better established dividing line, this size of sieve is used to separate the two materials. The test is sufficiently accurate and dependable even in the hands of different operators provided the sieves are of uniform mesh and size of wire and the sand is dry enough to be screened clean. The sieve mesh and size of wire should be standardized. In computing the proportions in which to use a given sand it is well to know how much of it will exceed 4-mesh in size and to increase the amount used in the concrete in the proper proportion and to correspondingly decrease the amount of coarse aggregate, as some of the coarse aggregate will be supplied with the sand.

Description of Material Retained on 4-Mesh Sieve is copied directly from the data sheet where it appears in space marked (12) and is entered in space (*j*) on the report form.

The purpose of this record is to furnish a general idea of the character of the material composing the sand on the assumption that the finer grains are of the same general character as these coarse pebbles which are much easier to examine and describe.

The test, if it may be called a test, has some value as a record of the general character of the stone making up the sand, especially in some cases in which the stone is of some unusual nature, or has some peculiarity worthy of note. Its usefulness in assisting in deciding whether a sand is suitable for a given purpose is limited to these unusual or special cases when it may give a clue to an undesirable characteristic of the material which might not otherwise appear until strength tests had been completed on the sand.

Mechanical Analysis of Material Retained on 4-Mesh Sieve is entered on the report form at (*k*), (*l*), (*m*), and the weight of the material sieved is entered at (*n*).

$$\text{Formula} \quad k = \frac{(13)}{(16)}. \quad l = \frac{(14)}{(16)}. \quad m = \frac{(15)}{(16)}$$

The sizes of the sieves used here are small, ¼, ½ and ¾ in. and do not cover the coarser sizes for the reason that a material submitted for examination as a sand seldom contains very coarse particles. The sieves used all have round holes and as the smallest is ¼ in. in diameter, there will be some material passing it which was retained on the 4-mesh sieve through which the entire sample as received was first passed, as the 4-mesh sieve has a rectangular opening about ⅛ in.

There is little reason for much variation in results in this test provided sieves are of accurate size.

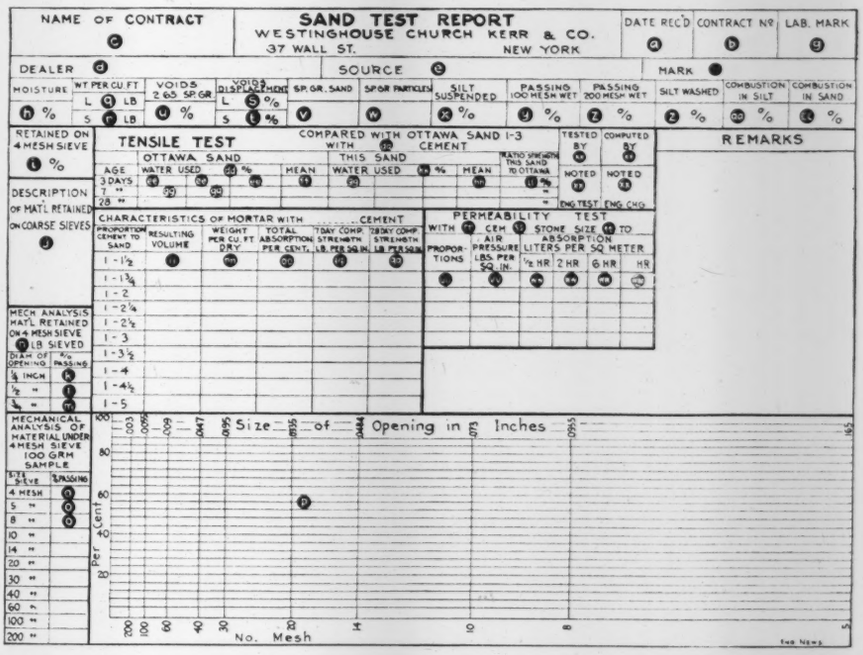


Fig. 2. Sand-Test Report. Office Report from Data in Fig. 1

(Letters in circles referred to in text.)

Mechanical Analysis of Material Passing 4-Mesh Sieve using a 100-gram sample, is entered in the report blank at (*o*), directly from the data sheet at (23). As the sample used weighed 100 grams the weights passing each size of sieve will read directly in per cent. without further computation. The results of this test are shown graphically on the report form by plotting the percentages shown at (*o*) as ordinates and sizes of sieve openings as abscissas on the specially ruled chart at (*p*).

The form of the curve as plotted on this chart gives, in the case of some sands, an indication of their quality. This indication is by no means positive, except when the sand goes beyond very wide limits as to size. Some sands whose analysis indicates a poor distribution of sizes show strength tests far in excess of others whose analysis would indicate a first-class sand. Of course a very fine sand or a very coarse one would be shown unsuitable by this test, but such a sand

can usually be judged by the eye without any test whatever.

The quality of a sand is dependent upon several characteristics, of which mechanical analysis or distribution of size of particles is but one. If the other controlling characteristics of two sands are equal, then their relative value might be roughly judged by their mechanical analysis. These conditions do not usually obtain, however, and it is therefore very risky to attempt to judge a sand on the results of this test. The test is of little primary value, but it is of considerable interest and of some importance in checking up the variation in sand from the same source. The accuracy of the test depends on the standardization of the sieves used and the thoroughness of the sifting.

WEIGHT PER CUBIC FOOT is determined twice: first, from the weight of a fraction of a cubic foot as shown on the data sheet at (17) and is recorded at (q).

$$\text{Formula:} \quad q = \frac{(18) - (19)}{(17)}$$

The measure is usually either ½ cu.ft. or ¼ cu.ft., depending upon the size of sample available, but in case the sample is quite small then a still smaller measure is used.

The second determination of the weight per cubic foot is made from the weight of 100 c.c. and is recorded at r on the report form.

$$\text{Formula:} \quad r = [(21) - (25)]\ \mathbf{0.624}$$

This second determination of weight per cu.ft. is only made for comparison with the first method, the data necessary being available on the data sheet for use in other tests.

This test is of doubtful value partly because of the great variation in results obtainable with the same sample depending upon the method and manipulation used in making the test. A variation of from 88.6 lb. to 107.8 lb. per cu.ft. has been obtained in the laboratory of Westinghouse Church Kerr & Co. on the same sample of sand and by methods in use in other laboratories. With such a variation possible this test can mean but little until the method of making it has become so standardized that results may be comparable. Even when standardized the results of this test have but little bearing upon the value of a sand for use in concrete.

PER CENT. OF VOIDS is also determined by more than one method. First, by weighing the amount of water which will fill the voids in the ½- or ¼-cu.ft. measure filled with sand which was used in getting the weight per cubic foot. The result is recorded on the report form at (s).

$$\text{Formula:} \quad s = \frac{[(20) - (21)] - [(18) - (19)]}{\mathbf{62.4} \times (17)}$$

The second method used consists in placing 100 c.c. of the sand into 100 c.c. of water and noting the volume of the mixture. The amount which their combined volume falls short of 200 c.c. gives the volume of the voids in the sand, and is recorded at (t).

$$\text{Formula:} \quad t = \mathbf{200} - (26)$$

A third method, which is in use in a number of laboratories, is based on the assumption that the sand has a specific gravity equal to that of pure quartz—namely 2.65. The voids are determined by computing the percentage that the weight of a unit volume of the sand falls short of that of the same volume of solid quartz having a specific gravity of 2.65. The result is recorded at (u).

$$\text{Formula:} \quad u = \frac{\mathbf{265} - [(24) - (25)]}{\mathbf{265}}$$

The value of this test is largely nullified by the lack of any recognized standard method of filling the container in which the sand is weighed. It is apparent that the voids in a sand are directly dependent upon the degree to which the sand is compacted. If a sample of sand is so loosely placed in the measure that 90 lb. of it will occupy a cubic foot, the voids will be greater than if the sample is so tightly compacted that 105 lb. are required to fill the cubic-foot measure. There is the same liability to error, and the range of error is as great, as in the case of determining weight per cubic foot pointed out above. If a standard test of reasonable accuracy were adopted this test would become a valuable one as the amount of cement required to fill the voids in a sand is directly proportional to the total volume of the voids. All other qualities being equal, the sand having the least voids is the best to use. But if the percentage of voids is determined by inaccurate methods the results on two sands may be enough in error to indicate the poorer of the two as the better one.

SPECIFIC GRAVITY OF SAND—This is but another way of expressing the weight per cubic foot and is found by dividing the weight of a measured volume of sand by the weight of an equivalent amount of water. The result is entered in the report form at (v).

$$\text{Formula:} \quad v = \frac{(24) - (25)}{\mathbf{100}}$$

This test has no particular value except as furnishing an interesting comparison between sands. It is open to the same severe criticisms as the tests for weight per cubic foot and percentage of voids; namely, that results are very largely dependent upon the degree to which the sand is compacted in the measuring vessel.

SPECIFIC GRAVITY OF PARTICLES—This test should be distinguished completely from the last described test which dealt with the sand as a body including the voids between the particles; whereas this test deals with the individual particles. The specific gravity is determined by dividing the weight of a quantity of sand in grams by its absolute volume in c.c. The specific gravity is entered in the report form at (w).

$$\text{Formula:} \quad w = \frac{(24) - (25)}{(26) - \mathbf{100}}$$

This test is of some value in showing up sands composed of light or soft materials which might be objectionable in first-class concrete.

SILT SUSPENDED—This test shows very roughly the amount of silt or very finely divided material in the sand by holding the fine particles in suspension in water while the coarser particles are settled out, allowing the fine silt to settle on top of the coarse grains. This is expressed as a percentage and recorded in the report at x.

$$\text{Formula:} \quad x = \frac{(28) - (27)}{(27)}$$

This test in some form or other is in very general use in examining sand. Often it is used in a very crude form, such as placing a handful of sand in a tall slender bottle and shaking it up with water and noting the result. It is a very fair indicator of the amount of fine material in a sand, and to one who has examined many sands it gives, in some cases, useful information. It is crude and inaccurate unless standardized, but if so standardized its usefulness would be increased.

PASSING 100-MESH WET—This is expressed in percentage of the sample screened and is recorded at (y) on the report form.

$$\text{Formula:} \quad y = \mathbf{100} - (35)$$

This test is but another form of the test for silt. It has been found that the sifting of a dried sample of sand often failed to show the true amount of fine material which should pass the 100- and 200-mesh sieves because the fine material adhered to the larger grains or formed masses of fine particles which were retained on the coarser sieves and figured in the results as coarser material. By washing a sample of the sand in a 100-mesh sieve under a full stream of water the fine silt is detached from the larger grains and the masses of fine particles disintegrated and washed through the sieve.

This test probably gives a truer indication of the amount of "fines" in a sand than does dry screening, but it may not be as valuable a test for silt or objectionable "fines" as some other form of test may be. A crushed rock, for instance, may show a high percentage of material passing the 100-mesh sieve and still be a first-class material for concrete.

PASSING 200-MESH WET—This test is similar to the

described except as to size of sieve mesh. The per cent of loss is recorded at (*z*) on the report form.

Formula: $z = \mathbf{100} - (36)$

This test is probably more useful than the one made with the 100-mesh sieve for the reason that fine material lying between the 200- and 100-mesh sizes is of service in obtaining density in concrete and if free from foreign matter is not objectionable if not present in excess. The material under 200-mesh probably carries more of the organic matter if there is any in the sand.

Silt Washed from a sample of the sand is also expressed as a percentage of the sample examined and is recorded at (*aa*) on the report form.

$$\text{Formula:} \quad aa = \frac{[(24) - (25)] - [(29) - (30)]}{[(24) - (25)]}$$

This method is one very commonly used in determining the amount of silt in a sand and has the virtue of being similar in a general way to the method that would be used in practice to free a sand of excessive or objectionable silt. Whenever sand is washed on a large scale the method used is more or less similar to that used in making this test. The results obtained in practice may not agree with those obtained in the laboratory, yet this test should give a fair indication of the amount of silt which might be removed by proper washing methods.

Combustible in Silt is expressed in percentage of the dry silt which has been washed from the sand. This percentage is recorded at (*bb*) on the report form.

$$\text{Formula:} \quad bb = \frac{[(31) - (32)] - [(33) - (34)]}{[(31) - (32)]}$$

Combustible in Sand is also expressed in percentage and is computed from the combustible in the silt on the assumption that all the combustible matter in a sand occurs in the silt. The result expressed as percentage is recorded at (*cc*) on the report form:

$$\text{Formula:} \quad cc = \frac{[(31) - (32)] - [(33) - (34)]}{[(24) - (25)]}$$

The value of these last named two tests is extremely doubtful. It may be that in the case of sands containing loam to an objectionable degree this test would be of service. There is as yet no established method of determining the safe limit of combustible matter in a sand. It is therefore a test which is of interest only, except possibly in a few rare cases. When sufficient data have been accumulated and tabulated and studied it may be possible to find the proper interpretation of the relation between the amount of loss on ignition of a sand and its value as an aggregate, provided such a relation exists. Until then the test has little general value.

Tensile Test—The ratio that the tensile strength, in pounds per square inch, of the sand under test bears to that of standard Ottawa sand when both are tested mixed with the same cement, in the same proportion and to the same consistency, is determined by this test. The cement used should, if possible, be of the same brand as will be used on the job. The identifying mark of the cement used is given on the data sheet at (49) and recorded on the report form at (*dd*). The three-day strengths of the three briquettes made of standard Ottawa sand are taken from the data sheet at (48) and recorded at (*ee*) on the report form. The mean of the three results is determined and recorded at (*ff*). In a similar manner the results of the 7- and 28-day tests with standard sand and the 3-, 7- and 28-day tests of the sand under test are taken from the data sheet directly at (48) and recorded on the report form at (*gg*). In each case the mean is calculated and recorded in the columns marked (*ff*) and (*hh*). The ratio or percentage that the strength of the sand under test bears to that of standard sand at each of the three ages is determined by dividing the mean strengths as shown in column (*hh*) by those shown in column (*ff*) and the results recorded in column (*ii*). The proportions used are one part of cement to three parts of dry sand by weight.

The percentage of water used in mixing the cement and standard sand briquettes is given on the data sheet at (45) and is recorded on the report form at (*dd*), while the percentage used with the sand under test is given at (46) and is recorded at (*kk*).

This comparative test of tensile strength is of considerable value in judging the value of a sand. Of those tests so far described it is probably the most useful one. A sand which develops a tensile strength of at least 70% of that of standard sand will probably give sufficient strength in compression when used in concrete for most of the ordinary construction work. For important work such as heavily loaded beams, girders, floor slabs and columns, a tensile strength equal to that of the standard sand, or nearly so, should be required. The test does not show conclusively what strength the sand will give in compression and as this is one of the most important qualities to be determined it fails to answer a very vital question. In many cases a sand may show only a small proportion of the strength of standard sand in the three-day test and on the seven-day test show a much higher ratio, and on the 28-day test it may equal or even exceed the standard. For this reason it is necessary to complete the longer time tests before judging a sand. This changing ratio is, however, useful in determining the relative time that forms should be left in place on the job. A sand which gains its strength slowly should be supported for a longer period than one which allows the cement to set up quickly. The question of the proper or safe time for removing forms from concrete construction is one that the engineer or architect should determine for the job foreman and this test made at the 3-, 7- and 28-day periods gives useful information on this important matter. The percentage of water required by a sand to give normal consistency in 1:3 mixture with cement is often a guide as to the quality of the sand. In most cases a poor sand requires much more water than a good sand to produce normal consistency. This is particularly true of fine sands. The relative percentage of water required as shown at (jj) and (kk) on the report form should therefore be noted in studying a report on sand.

Resulting Volume of Mixtures of cement and sand is shown on the data sheet for several proportions of mixture in column marked (39). These are recorded on the report form in column marked (*ll*) in their proper lines. The test shows the increase in volume of a measured amount of sand when a definite proportion of cement has been added to it and the whole mixed with water to form a mortar. The laboratory mark of the cement used as given on the data sheet at (37) is recorded on the report form at (*mm*).

This test indicates in a practical manner the amount of cement required to fill the voids in the sand and furnish a safe excess to insure the voids being filled. The methods of manipulation in this test are important and if the test proves of sufficient value these methods should be standardized.

Weight per Cubic Foot, Dry, of Cement-Sand Mortar is computed from the weight and volume of the 2-in. cubes made of varying proportions of cement and sand which were mixed for the "Resulting Volume" test. The weight of the dry cube is shown in grams on the data sheet in column (44) and the volume in cubic centimeters is found by deducting from the weight wet given in column (42) the weight of the cube suspended in water given in column (43). As all these weights are in grams and as water is the fluid used in getting the immersed weight, the difference between the weight of the cube in air and the weight in water represents the cubic centimeters in the volume of the cube. As the cube was saturated with water before weighing, there is no absorption of water taking place when the specimen is immersed for weighing, so the displacement is equal to the gross volume of the cube including the voids which are filled with water. The weight of the dry cube divided

by the displacement gives the specific gravity of the specimen, which, multiplied by the weight of a cubic foot of water, gives the weight per cubic foot of the dry mortar, which is recorded on the report form at (*nn*).

$$\text{Formula:}\quad nn = \frac{\mathbf{62.4} \times (44)}{[(42 - (43)]}$$

This test is a valuable check on the other tests of the various mixtures of cement and the sand under test. The densest mortar should give the strongest and least permeable concrete. While no rules or formulas are now available to determine other qualities of a mortar from its weight per cubic foot, yet there seems to exist a fairly definite relation between density and compressive strength when the same materials are used. It is possible that if sufficient investigative effort were applied to this particular subject some valuable data might be accumulated, a study of which might lead to the formulation of rules which might be used in specifying the proportions of cement and sand to be used on the work in hand.

TOTAL ABSORPTION is expressed as a percentage of the weight of the dry mortar, and the result expressed in percentage, is recorded on the report form in column marked (*oo*).

$$\text{Formula:}\quad oo = \frac{[(42) - (44)]}{(44)}$$

This test is very closely allied to the last test; namely, weight per cubic foot, when the same materials are considered, as both are forms of expressing the degree to which the specimen approaches a solid body or maximum density, the weight being directly and the absorption inversely proportional to the density. This test is of value when the work in hand is one requiring a definite degree of absorption as, for instance, a mortar exposed to water and frost in a severe climate where a mortar having above a certain percentage of absorption will be damaged by the repeated freezing and thawing of absorbed water.

SEVEN-DAY COMPRESSIVE STRENGTH is expressed in pounds per square inch, the area of the specimen being 4 sq.in., and is recorded on the report form in column marked (*pp*).

$$\text{Formula:}\quad pp = \frac{(40)}{\mathbf{4}}$$

TWENTY-EIGHT-DAY COMPRESSIVE STRENGTH is similarly expressed and recorded at (*qq*).

$$\text{Formula:}\quad qq = \frac{(41)}{\mathbf{4}}$$

These two tests, the seven- and the 28-day compressive tests, made on a progressive series of mixes, are by far the most valuable tests to which a sand, which is to be used in important reinforced-concrete work where strength is one of the first essentials, may be subjected in the laboratory. A series of compression tests gives the only definite data on which to base an answer to the one most vital question which should be asked by whoever is responsible for the mixture to be used on a concrete job. The all important question which arises when the proportions for a concrete designed for a definite strength are to be determined is "How much cement must be added to the sand to give the required strength?" The answer to this question should form the basis of all decisions as to mixtures to be used when strength is the prime consideration. Compression tests on a series of mixtures of progressive richness in cement gives the only direct and strictly relevant data on which to base a decision on the question of what mixture to use in a given case. It is very important that a test of this nature be standardized and that the details of manipulation used in making the specimens be very carefully worked out to the end that results obtained by different operators in different laboratories may be comparable. Such details as method of mixing, consistency of the mixture, method of filling and molds and tamping, and the method of storing the specimens, have a great influence on the results obtained and they should be so standardized that reasonably uniform results may be obtained.

PERMEABILITY TEST is applied in those cases where the sand is to be used in making water-tight structures. The laboratory mark of the cement used in making up the specimens is taken from the data sheet at (50) and recorded on the report form at (*rr*). Also, the laboratory mark of the stone used and its size is copied from the data sheet at (51) and (52) on to the report form at (*ss*) and (*tt*).

The proportions of the various mixtures used as given in the column marked (53) on the data sheet, are recorded in column (*uu*) on the report form. The air pressure used is recorded in column (*vv*) from column (56) of the data sheet.

The absorption in liters per square meter is computed from the readings of the water column scale of the apparatus as given in columns marked (60) on the data sheet by means of a conversion factor. The test is performed on an area of 57.5 sq.cm. or a 3⅜-in. diameter circle and each division on the water column of the apparatus represents 3.45 c.c. By multiplying the scale readings by 174, which represents the number of 3⅜-in. circles in a square meter, and by 3.45, the number of cubic centimeters represented by each division of the scale, and dividing by 1000 by pointing off three places, the result expresses the liters which would be absorbed by a square meter under the same conditions. This result is recorded on the report form for the half-hour, two-hour, six-hour and in some cases, longer periods in the columns marked (*ww*).

$$\text{Formula:}\quad ww = [(60) - (59)] \times \mathbf{0.6}$$

The value of this test in those cases where impermeability is the chief consideration is equal to that of the compression tests of a series of mixtures in those cases where strength is of the first importance. With these two series of tests carefully carried out under standard conditions there are supplied data on which to base decisions not only as to the suitability of a sand for a given purpose, or the relative value of two or more sands, but also the proper proportions in which to use the sand selected to produce the desired result.

ADDITIONAL MATTER—In addition to the above data and results there is recorded on the report form the initials of the parties to the tests and computations and of the Engineer of Tests and the Engineer in Charge of the job on which the sand is to be used. The initials of these four persons are placed in the four rectangles marked (*xx*).

Remarks and specifications for use are recorded in space at the right of the report form headed "Remarks" and marked (*yy*).

The Application of Brick in Engineering Work*

BY E. E. R. TRATMAN†

The use of brick in engineering work is extensive and varied. In recent years its use has become considerably restricted in some lines of construction, while some new lines have become important.

BRICK FOR SEWER CONSTRUCTION

The sewer systems of cities afford one of the largest fields for the use of brick. Much of the work is of an ordinary character, but there are many cases in which size, form, special design, methods of construction, or difficulties encountered, make the work of interest to the engineer. At Detroit, most of the sewers are of brick. The largest brick sewer planned and built during 1913 was the Morrell St. sewer, about five miles long, from 8 to 9 ft. diameter, and costing $685,000. A similar trunk sewer now under construction is the Mt. Elliott Ave. sewer, a considerable portion of which is 9 ft. diameter, built in tunnel.

At Cincinnati there are about 80 miles of brick trunk

*Abstract of a paper presented at the annual convention of the National Brick Manufacturers' Association, at New Orleans, La., Mar. 2-7, 1914.

†Resident Editor of "Engineering News," Chicago.

...rs. Brick is used for all sewers over 2 ft. diameter, and the largest brick sewer is about 20 ft. diameter. The larger ... and the smaller ones where necessary) are backed up ... haunch walls of rubble masonry or concrete. Vitrified-... lining is used in the inverts of concrete sewers (up to ... springing line) but only where the velocity of flow is ...essive; that is, over 8 ft. per sec. In some cities, however, ... concrete sewers are lined with brick (at least in the in...), while others do not use brick in any of their concrete sewers.

At St. Louis, brick is used for all sewers over 27 in. diameter and up to 9 ft. diameter, and is used for still larger sizes in special cases. During 1913 there was under construction the Rocky Branch joint district sewer, 10 and 14 ft. diameter, brick being used because the congested condition of the streets made the establishment of a concrete plant a difficult matter. There was also the Glaise Creek joint district sewer, a trunk sewer of 10 ft. down to 8 ft. diameter, brick being used entirely because the amount over 9 ft. diameter was too small to warrant installing a concrete plant. Vitrified-brick lining is used in the invert of concrete sewers.

At Philadelphia, brick is used in egg-shaped sewers 2 ft. 3 in. by 1 ft. 6 in. to 4 ft. 6 in. by 3 ft. and in circular sewers from 2 ft. to 20 ft. diameter. Concrete is used only for sewers up to 5 ft. diameter except in special cases, and has brick lining in the invert only.

In Chicago there are about 1958 miles of sewers, of which all but 14½ miles are of brick, with a diameter of 3 ft. to 10½ ft. The largest concrete sewer is of elliptical section 14 ft. high and 12 ft. wide. No brick lining is used in any of the concrete sewers in Chicago or Kansas City.

At Cleveland, all but about 6% of the large sewers are of brick, the smallest size being egg-shaped in section, 24 in. wide and 27 in. high, and the largest being 16 ft. diameter. Concrete is used to a considerable extent for the foundations of sewers, and in several cases there has been used a combination of brick lining for the entire inner ring, backed with concrete.

At Minneapolis, brick is used only for the manholes and catchbasins. At Kansas City, no brick sewers have been built for three years. For sewers over 30 in. diameter, bids are asked on brick, reinforced monolithic concrete, and reinforced-concrete pipe. The awards are made to the lowest price material, which is generally monolithic concrete, but in some cases concrete pipe. The concrete sewers are not lined with brick. A special development is the use of hollow segmental blocks, and, owing to their size and light weight, the work can be done much more rapidly than with brick. These blocks have been used in sewers from 27 in. to 6 ft. diameter, and are made for sizes up to 9 ft. diameter.

BRICK TUNNEL LINING

At Chicago, brick has been used in certain parts of the Chicago Ave. and Wilson Ave. tunnels of the water-works system; in all about ½ mile of tunnel, 8 ft. diameter. The water-intake tunnels at Cleveland have a brick lining, and brick is to be used also in the new intake tunnel at Milwaukee. At Cleveland, a long intercepting sewer has been built parallel to the short line, and nearly five miles of this are in tunnel with a four-ring brick lining, having an interior diameter of 13½ ft. and 12¼ ft. This was driven in stiff clay soil, by means of a steel shield. The bricks were laid in portland-cement mortar, and though the jacks pressed against the end of the completed lining, the only damage done was at the points of application of the jacks, and this small amount of masonry was repaired each day as the shield advanced.*

The lining of railway tunnels was at one time a field for brick, but this is now much less extensive. An example of modern work is the double-track Elm tunnel on the Frankfort & Berlin Ry. in Germany.† This is 2¼ miles long, and was built under considerable difficulties, owing to the pressure or "squeeze" of the ground. The celebrated Thames tunnel was built of brick, as were the large double-track arched tunnels of the original underground railways at London. These were near the surface, but the present "tube" lines are single-track deep-level tubes with cast-iron lining. With the advent of the modern subway and underground railway there has been opened a rather large field for the architectural or decorative use of brick in the facing of the tunnel lining at the stations, and, though this is mainly for glazed and ornamental brick, it indicates how new fields open, although old fields for the use of brick may become closed or restricted.

CHIMNEYS

Most important brick chimneys now built are of special radial hollow brick. The monster brick chimney for the Boston & Montana Copper Co., at Great Falls, Mont., is 506 ft. high and 50 ft. diameter at the top, the proportions being very unusual. It is built on the Custodis system, and is said to be both the tallest and largest chimney in the world. Next to it comes a Heinecke brick chimney 460 ft. high, but with a diameter of only 8 ft. at the top. This chimney carries the acid gases from smelters and is reported to show no deterioration after 20 years' service.

Another exceptional brick chimney, which is large in diameter as well as in height, is that of the St. Joseph Lead Co., at Herculaneum, Mo. This is 350 ft. high and 20 ft. diameter at the top. It has an octagonal base 40 ft. high and a circular shaft 310 ft. high. The base contains two openings and is built of vitrified paving brick, laid in English bond. The shaft is built of the Heine perforated interlocking radial brick, and the walls are laid up with alternate header and stretcher courses. The chimney is reinforced by steel bands 4x½ in., embedded in the brickwork, and spaced about 20 ft. apart for the entire height. Only the lower part of the interior is protected by a lining (40 ft. high), owing to the relatively low temperatures of the entering gases. Both the paving brick and the radial brick were hard-burned shale brick, having a compressive strength of over 5000 lb. per sq.in. and with a very low absorption. They were tested for their acid-resisting qualities by immersion in 25% sulphuric-acid solution for 48 hr. and afterward by boiling this solution continuously for 30 min. without removing the sample from the solution. No effects of the acid on the brick were noted.

The chimney was built to carry off the products of combustion and the fumes of a lead-smelter plant. The gases have a temperature of about 200° F. and contain a high percentage of sulphur, making it necessary to build the chimney of acid-proof materials. The bricks are laid in lime-cement mortar, on account of its strength, but as this mortar is decomposed in time under the influence of the acid in the gases, all the mortar joints liable to be attacked by the contact of the gases were made acid-proof. The interior joints for the entire height of the chimney and the exterior joints for 100 ft. down from the top were raked out to a depth of 1½ in. and pointed up with a mortar made of raw benezet. The entire lining was laid with this mortar. The material is a very high grade of fireclay and is acid-proof; it contains about 36% alumina and 49% silica, with only a trace of lime, magnesia and iron oxide. The top of the chimney is covered with a heavy lead cap, extending 12 in. on the inside and outside, and for 100 ft. from the top the iron ladder rungs are covered with ⅛-in. of lead. Quite recently a brick chimney has been built at a blast-furnace plant, serving the hot-blast stoves, which is said to be the first used for this particular purpose. At industrial plants there has been a tendency to use steel chimneys serving one or two boilers each, due to a certain prejudice against a single large central brick stack, but there is a much greater loss of heat through the steel stack, due to radiation and convection, unless there is the additional expense of a brick lining.

BRICK FOR PAVING

The use of brick for paving is a matter of great engineering importance, and in the past 15 or 20 years the manufacture of paving brick has developed into a special industry. Engineering experimental and research or laboratory work has assisted largely in this development, in various ways: 1, the investigation of clays and their properties; 2, the manufacture of the brick, so as to secure a tough, hard and uniform product; 3, the testing of the finished product; and 4, the proper methods of constructing the pavement. For many years, brick paving was confined mainly to the streets of towns and cities but it is now being applied extensively to the paving of country highways.

BRICK FOR BRIDGES AND DOCK WALLS

At one time brick was extensively used for bridges and large arches, but this use has become practically obsolete in this country and is on the decline in other countries. However, in some arch bridges of concrete construction brick is used as a facing, in order to produce an effective appearance and to form a durable surface finish. This has been done more extensively abroad, but one example in this country is the Sixth Ave. bridge in Des Moines, Iowa, which has a series of arches, with red-brick facing on the arches and piers.

Another use of brick which is more common abroad than in this country is the facing of the walls of docks and piers. Concrete forms the mass of the wall, but this material is often liable to disintegration, particularly at the water line. It is also difficult to repair when worn or when the surface is broken by heavy blows or chafing. The brick used for this facing, in foreign work, are usually of a special character, much larger than ordinary brick, extremely hard, and frequently dark blue in color. At points on the Nova Scotia coast walls have been faced with vitrified brick.

*"Engineering News," Mar. 28, 1912.
†"Engineering News," Dec. 11, 1913.

Condition of the Montreal Water Conduit, and Protective Measures

The Board of Engineers appointed to investigate the Montreal water conduit break, of Dec. 25, 1913, which cut off the city's water-supply for eight days, has just submitted a report on the condition of the conduit and the circumstances attendant on the break. The members of the board are J. A. Jamieson, R. S. Lea, and G. R. Heckle. They find that *the conduit is in bad condition*, that its design is inadequate, that the work incident to the adjacent canal enlargement damaged the conduit, and that the plans for the canal enlargement do not assure the security of the conduit. The report is printed below (slightly abbreviated).

Our account of the conduit break (ENG. NEWS, Jan. 1, 1914, p. 51) also described the conduit and the canal ("aqueduct") along whose north bank the conduit is located. The conduit was built 1907 to 1909, by P. McGovern, and immediately on its completion Quinlan & Robertson began work on enlargement of the canal to such size as to allow of developing 2500 hp. hydraulic power at the lower end, the water-works pumping station. This enlargement work was not quite completed before the city authorities decided to enlarge for still greater power capacity to take care of street lighting in addition to pumping. Contract for this second enlargement was let to the Cook Construction Co., July 17, 1913.

The second enlargement includes the construction of retaining-walls on both sides, whereas the first enlargement was to be for sloping sides. Partly for this reason, the excavation in the second enlargement came much closer to the conduit. As noted in our account of Jan. 1, a drag-line excavator operating on top of the bank, over the conduit, in taking out the canal excavation caused a slip of earth that exposed the side of the conduit near the asylum bridge at Verdun (Sta. 123 to 124, counting from lower or pumping-station end). There was some leakage from the side of the conduit. Several days later, on Dec. 25, the lower half of the exposed side blew out, for a length of 60 ft.

The inspection was made within the week following the break. As the conduit has not been empty since first put into service—being the sole line supplying Montreal with water—there has been no prior inspection.

CONDITION OF CONDUIT

The concrete was found to be of good quality and workmanship throughout the entire length. The conduit from the intake end at river (Sta. 270 + 50) to Sta. 220 was found to be practically free from cracks or other defects. Along this part of the conduit the depth of the overlying earth is only 6 to 8 ft.

At about Sta. 220, there is a coffer-dam built across the aqueduct. From this point to the river the water stands in the aqueduct at the river level, while from the coffer-dam eastward to the pumping station the aqueduct has been unwatered to permit the enlargement to proceed.

Between Sta. 220 and 214, a distance of 600 ft., a serious crack was found in the crown of the arch. This crack was about ¾ in. in width, and in places the roof on one side has sunk down about ¾ in further than the other. There are also corresponding cracks in the invert, and signs of strain along the spring line on both sides. These cracks clearly indicate a distortion of the conduit from an overlying load.

An inspection of the exterior disclosed the fact that along the cracked section excavated material had been dumped directly over the conduit up to about El. 58, making the overload about 25 ft. above the crown, plainly accounting for the cracked and distorted conditions found. The east end of the cracked section corresponds with the point where the conduit passes from earth into rock.

A subsequent inspection along the aqueduct bank showed a very considerable leakage of water from the conduit along the cracked section between Sta. 220 and 214. For the most part this water finds its way into the aqueduct through gravel strata, and shows no signs of transporting any of the clay material; and the conduit is located at this point approximately 40 ft. back from the aqueduct bank.

Between Sta. 214 and 133 the conduit is located partially or wholly in rock, and was found to be generally in good condition.

Between Sta. 133 and 120 there are cracks along the crown, both sides, at about the springing line of the arch, and in the bottom, but these are as a rule not open to any extent. They, however, indicate considerable stress and some distortion in the conduit. It was in this section betwen Sta. 123 and 124 that the break occurred, and an inspection made after the conduit was filled with water disclosed the fact that at several points along the bank water was leaking from the conduit and finding its way into the aqueduct.

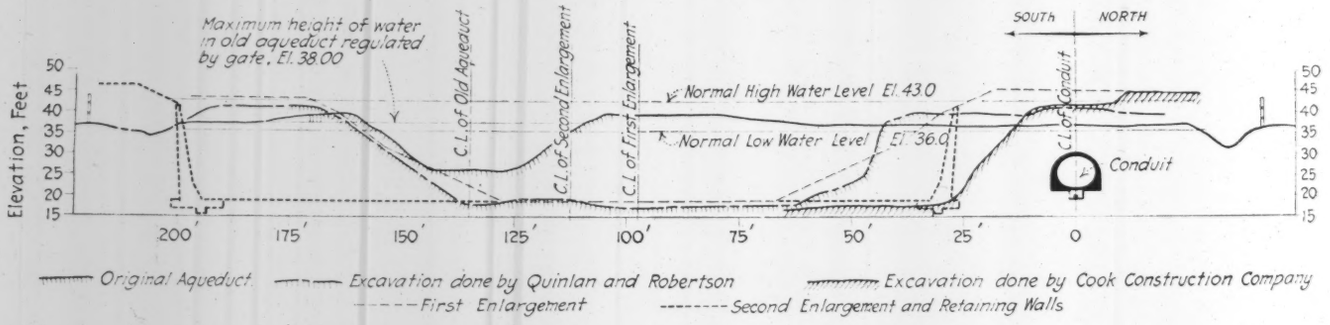


FIG. 1. MONTREAL AQUEDUCT ENLARGEMENTS AND CONDUIT. COMPARATIVE SECTIONS AT STA. 122.

Between Sta. 69 and 120 there is a light crack along the crown for a considerable part of the distance, but for the most part this crack is comparatively small. An apparent leak has been found in this section near Sta. 94, but the water was running out through gravel strata and was quite clear, indicating that the leakage is probably not endangering the conduit at this point.

Between Sta. 69 and 50, the conduit is cracked along the crown and in the invert, these cracks being open for the most part about ⅛ in., and by cutting into the concrete at certain points it was found that the reinforcing wires were broken in the crown. There are also some cracks at about the springing line of the arch. Along this section it was found that the original fill over the conduit was only about 5 ft., but that, during the operations under the Quinlan & Robertson contract, excavated material from the aqueduct had been deposited along the bank over the conduit up to approximately El. 45, making the fill about 15 ft. higher than the top of the conduit. This fill is, however, not generally over the center of the conduit, but south of it, thus producing an unbalanced loading. A close examination would indicate that these cracks were not of recent date.

Between Sta. 50 and about 30 the conduit was in good condition, showing no evidence of anything beyond occasional fine cracks. It was impossible to proceed further east with the inspection on account of the depth of water.

An inspection was made of the existing headgates at the entrance of the aqueduct, and where the section has not been disturbed by either contracts Nos. 1 or 2. It was found that the aqueduct had been provided with both headgates at the entrance from the river and regulating gates further down,

but gates in both cases the gates were in very bad repair, and entirely useless at the present time.

That there should be a means available at all times for controlling the flow of water in the aqueduct, we have urged that the Cook Construction Co. proceed at once with the construction of the coffer-dam at the entrance as covered by their contract, also that the necessary gates and sluices for controlling the flow be included in the structure.

EMERGENCY MEASURES

On the night of Dec. 30, after completing the above inspection of the interior of the conduit, we verbally made certain recommendations, as follows:

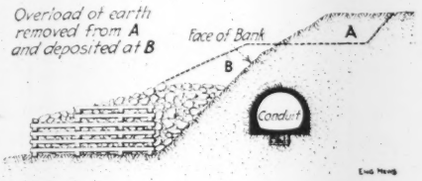


FIG. 2. EMERGENCY SUPPORT OF NORTH BANK, MONTREAL AQUEDUCT CANAL, TO PROTECT CONDUIT

1. That riprap be placed against the toe of the slope along the weakened sections in the vicinity of the break as rapidly as possible.

2. That, when the water was turned into the conduit, it should not be allowed to run more than three-quarters full and kept below that depth if possible.

3. That the overload of earth be removed from the top of the conduit for a considerable distance east and west of the break.

4. That the overload on the section lying approximately between Sta. 214 and 220 be also removed as soon as possible.

5. That the excavation work on the north side of the aqueduct be suspended, and that for the present no blasting be allowed in the rock sections on the north side.

These verbal recommendations were confirmed and further emphasized in a written report [see "Eng. News," Jan. 22, 1914, p. 215] in which certain other additional recommendations were made, including the construction of a rock-filled timber crib along the toe of the bank at the weakened section both east and west of the break, to support a fill in an endeavor to reproduce the original conditions of loading.

In accordance with the recommendations of the above report, an arrangement was made by your Board with the Cook Construction Co. to proceed immediately with the carrying out of the work involved. The most important measure, namely, placing riprap along the toe of the slope at the weakened section on both sides of the break, between Sta. 119 and 129, to be followed at once with the construction of a rock-filled crib, was proceeded with first, night and day shifts being employed. J. R. Dorrance was placed in charge of the work by the city as resident engineer, and F. S. Pearson as superintendent.

While the above-mentioned work was being carried out, the very difficult matter of securing the conduit at both ends of the steel pipe at the break was being done. This work required the utmost care, as it was found that cavities had been eroded on both sides of the conduit and directly underneath by leakage, which, of course, made the situation a very serious one. Blocks of concrete were placed in sections underneath the conduit where the foundation was scoured out, and the leaks in the side were calked and afterward incased in concrete. The water draining through the conduit in both directions toward the break and at other points on both sides was carried in troughs to the aqueduct, so that no further scouring could occur.

Fig. 2 shows a cross-section through the rock-filled crib and the conduit, and gives a general idea of this work.

When the protection work adjacent to the break was well under way, a force was started removing the excessive load from the top of the conduit between Sta. 214 and 220, a wedge-shaped section being taken off to an elevation of approximately 47, thus removing about half the load over the crown. It was considered advisable to do this work entirely by hand labor, on account of the danger of using machinery adjacent to this cracked section.

At this time it was thought advisable by your Board, and with the approval of John R. Freeman, to lay a pipe connecting the Lachine Canal with the present pump-well; this work being at once proceeded with. In our opinion this precaution is justified by the possibility of another break in the conduit which might expose the city to the danger of a conflagration. We believe, however, that this should not interfere in any way with measures being taken as soon as possible for furnishing the city with an alternative supply of pure water, and that water from the Lachine Canal should not be turned into the city mains except as a last resort.

DESIGN AND REINFORCEMENT OF THE CONDUIT

Fig. 3 shows a cross-section of the conduit used in soft soil. The dotted line below the springing line of the arch shows the form used in firm soil or rock. In both the latter cases, the reinforcement was to be omitted below a point 12 in. below the springing line. In firm soil, the earth was to be excavated to the neat line of the bottom and the side walls. In rock, any excess excavation was to be filled with lean concrete composed of 1 part of cement, 3 parts of sand and 8 parts of 3-in. broken stone.

Article 18 of the specifications covers the character and location of the reinforcement. It reads as follows:

18. The reinforcing material of the conduit shall consist of expanded steel No. 6 with 6-in. mesh, or of other reinforcing metal which shall be known to have an equal degree of resistance. In the case of tenderers wishing to substitute a different material than No. 6 expanded steel with 6-in. mesh, they will have to clearly specify the nature and weight of the reinforcing material they propose to make use of. The reinforcing metal shall be set at about 3 in. from the interior surface of the conduit, and each sheet of metal shall be lapped over the adjoining one, to the width of one mesh to form a continuity of joints, but only in the crown of the conduit shall the adjoining sheets of metal be attached together with wire.

As will be noted in Fig. 3, Clinton wire cloth was substituted for the expanded metal specified.

With regard to the design of the conduit: From the information which we have obtained from the city, the maximum depth of fill over the crown would be approximately 25 ft. Engineering practice in concrete conduit design exhibits a considerable variation in the sections of concrete and the percentage of reinforcement used. While there are examples of as light, or possibly lighter, types of similar structures in existence, we consider the concrete sections employed in the Montreal conduit are lighter than are warranted by safe engineering practice. We feel, moreover, that the percentage of steel used and the position in which it is placed does not constitute an adequate reinforcement against the stresses to which the structure is subjected. The above statements are particularly pertinent having regard to the fact that this conduit represented the only water-supply line to the city.

The weight per square foot of the reinforcement called for in the specifications is not stated therein, and, as expanded steel of this particular class is not now commonly manufactured, we have not been able to definitely compare the relative weights of the steel fabric specified with that actually used. So far as our information goes, however, it appears to us that the reinforcement used does not represent an adequate equivalent for the expanded steel referred to in Art. 18.

So far as our inspection of the conduit enables us to report on the matter, we consider that the concrete is of good quality, and that in general the work has been well executed.

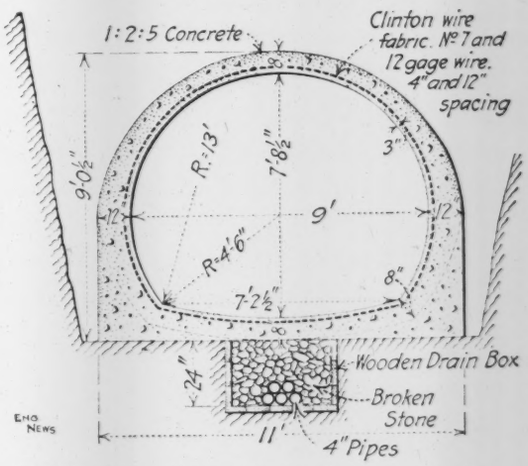


FIG. 3. TYPICAL SECTION OF MONTREAL CONDUIT. (DOTTED LINE BELOW SPRING OF ARCH IS FOR ROCK AND FIRM EARTH)

The soil through which the aqueduct is being excavated and in which the conduit is embedded for the greater part of its length, is largely composed of boulder clay or rock powder. When comparatively dry, or well drained, it will stand in a bank with practically vertical face, but when fully saturated with water tends to flow to a horizontal surface, and is

easily eroded and transported by running water. This latter property has a very direct bearing on the safety of the conduit, in view of the proximity of the aqueduct excavation.

Water leaking from the conduit under pressure tends to find an outlet through the intervening bank into the adjacent aqueduct, the bottom of which is about 5 ft. lower than the invert. This water erodes and transports the soil surrounding the conduit, removing its lateral support, and the leaks are a serious source of danger, so long as the aqueduct remains unwatered.

The only practical way to stop these leaks, in our opinion, is by filling the cracks from the interior. This, however, can only be done by unwatering the conduit. Under existing conditions, however, this latter cannot be done within a reasonable time owing to the fact that there is only one 12-in. pipe through which the conduit can be drained. The suction pipes of the pumps at the pumping station do not extend sufficiently low by several feet to permit the conduit being unwatered by pumping. We would therefore recommend that the installation of a new suction well or gallery, at least 6 ft. lower than the existing one, should be immediately constructed, and the suctions correspondingly lowered . This arrangement would permit the conduit to be quickly unwatered, and the work of interior repair proceeded with during short periods, and would also permit the conduit being run partially full of water.

THE CANAL RETAINING-WALLS

Referring to the contract of the Cook Construction Co. for the widening and deepening of the aqueduct, we find that the contractor is required to construct reinforced-concrete retaining-walls along both sides of the aqueduct in the earth section, and plain concrete retaining-walls in the rock section.

While the contractor is, in his contract, called upon to use extreme care and diligence in his work in order not to endanger in any way the water-supply conduit, and is particularly cautioned regarding the sections between Sta. 10 and 14, and from 80 to 85, we do not find that there is any particular provision made by which the contractor is obliged to follow up his earth excavation immediately with the construction of the retaining walls. Moreover, the contract provides that the contractor construct test sections of the wall when ordered by the engineer and before proceeding with the general construction. This would seem to indicate that the city engineers did not expect the contractor to follow up his earth excavation very closely with the retaining-walls. It also seems to indicate that the city engineers themselves were not quite satisfied that the wall as designed was sufficiently strong for the purpose intended.

It is obvious from what has already occurred and from the present conditions of the north bank in the earth section, that unless the wall construction followed the excavation very closely, the conduit would be greatly endangered, as there are points east of where the break occurred where the plans show that the excavation was to be carried much closer to the conduit than at points already excavated.

As one of our recommendations, made immediately after our first inspection of the conduit, and confirmed in our interim report, was to the effect that all work on the north side of the aqueduct should be suspended by the contractor for the time being, we have felt obliged in preparing this report to make an analysis of the design of the reinforced-concrete retaining-wall which the Cook contract provides shall be built in the earth section. From our analysis we have found that the wall would not be stable for the loads which we believe it prudent to assume it would have to sustain.

Another apparent weakness in this design is that the bottom of the concrete base is only 1 ft. 9 in. below the bottom of the canal. If the structure were left exposed during construction in the winter time, or if, at a later period, the canal should be emptied of water in the winter time, there would, we believe, be great danger of frost penetrating under the base, causing serious damage to the wall.

From a review of the features which have been described in this report, we have arrived at the following conclusions:

1. That the plans and specifications of the conduit did not call for sufficient reinforcement, nor is the steel properly placed in the concrete section, to provide a sufficient factor of safety, especially in view of the fact that the conduit would be the only means of supplying the city with water during the enlargement of the aqueduct.

2. That in the construction of the conduit the quality of the concrete and the workmanship generally were good, but that the weight and strength of the steel reinforcement was much less than called for by the specifications submitted to us.

3. That the insecurity of the conduit was greatly increased by the proximity of the excavation involved in carrying out Enlargement No. 2.

4. That the method of performing the work by means of a heavy drag-line excavator traveling on top of the bank above the conduit imposing a heavy vibrating load concurrently with the removal of the earth which provided lateral support to the conduit further endangered the structure.

5. That the leakage of water from the conduit is an additional source of danger at points where the conduit is located in easily eroded soil, and the excavation carried sufficiently close to permit the water finding an outlet into the aqueduct.

6. That the failure of the conduit was due to the proximity of the excavation, to the methods employed in the execution of the work, and to the inherent weakness of the conduit itself.

7. That the conduit, as a permanent means of supplying water to the city, is not now to be depended upon.

8. That it would be unsafe to proceed with any further excavation in the earth section on the north side of the aqueduct with the conduit in its present condition and while there is no other water-supply available.

9. That it is unsafe to do any blasting in the rock section on the north side of the aqueduct.

10. That it is unsafe to allow the traveling excavators or locomotives or any other similar heavy machinery to be operated on or moved at any point over the top of the conduit.

11. That the reinforced-concrete retaining-wall as designed by the city and to be built by the Cook Construction Co. along the banks of the aqueduct in the earth sections is not, as designed, a safe structure to build for the purposes intended.

12. That a revision of the design of this wall, in order to make it safe, will greatly increase the cost of the project.

RECOMMENDATIONS

With the above facts as a basis, we beg to make the following recommendations:

1. That before any further work is proceeded with, at least on the north side of the aqueduct, an investigation be made by a commission of engineers into the entire aqueduct scheme, which will include revised estimates of the cost of construction, and the quantity and cost of the power developed.

2. That the city at the earliest date possible make arrangements for providing a new and independent water-supply of a suitable character, so located that it cannot be affected by any further accident which might happen to the present conduit.

3. That a permanent coffer-dam or intake be at once constructed at the entrance to the aqueduct including proper gates or sluices that may be required, so that the flow of water into the aqueduct may be at all times under control.

4. That until another suitable water-supply has been provided the present one should be kept continuously under observation so that in case the present leaks become serious prompt remedial measures may be taken.

5. That the suction pipes of the pumps at the pumping station be lowered by at least 6 ft. as soon as possible so that the conduit may be run partly full or may be quickly emptied.

Automatic Sprinklers for Fire-Protection on Ships

A certain amount of attention has been given in recent years to the problem of applying to ships the same principles of fire-protection that have been found effective in reducing the fire loss in factory buildings. Several ships are now equipped with sprinklers, but in the large total of shipping they are only rare exceptional cases. Generally speaking, ship owners and others interested in shipping regard the idea of installing sprinklers as remote and radical.

A few months ago, C. H. Phinney, of the Inspection Department of the Associated Factory Mutual Fire Insurance Cos., Boston, presented a brief on the subject of fire prevention and fire-protection on ships to the U. S. Commissioner of Navigation as a preliminary to the International Conference on Safety at Sea. This brief discussed such matters as steel construction, elimination of wood in trim and other minor elements, subdivision of space to restrict fires, the reduction of fire hazard from

machines and electric equipment, etc., and laid strong emphasis on the matter of sprinkler installation. The following passages dealing with sprinkler equipment are pertinent.

Automatic sprinklers have now become so common on shore that description is unnecessary. Not only are they found in nearly all mills, factories and warehouses, but in thousands of large stores, halls, theaters, etc., throughout the country. Soon they will be compulsory in cities.

We believe there is no reason why they cannot be adopted to the protection of ships at sea. That the adoption of this protection in ships would immeasurably reduce the fire hazard cannot be doubted. The question is, does this danger justify the adding of this apparatus to the fire-protecting equipment at least of large passenger vessels? To my mind, there is but one answer—Where the decks and upper works are wood, provide sprinklers everywhere. Even in all-steel vessels I should advocate their introduction over such compartments as have specially hazardous occupancy.

In installing a sprinkler system in a ship the heads would be spaced about 10 ft. apart, each head protecting about 100 sq.ft. The cost of this protection would probably average about $4 per head, not including pumps. In our practice it costs less than $3, but I am assuming that owing to special and broken construction it might cost more on shipboard.

Shortly after this brief was written there occurred the fires on the "Imperator" and on the "Volturno." The following additional remarks by Capt. Phinney bear on these two instances.

Referring to the "Imperator" fire, he says:

Every incident connected with this fire, as reported by the press, indicated that had sprinkler protection been provided there would probably have been no fire to speak of, no smoke to produce a panic, and but very little loss on ship's cargo. This fire started far down in the bottom of the ship, a locality that was very difficult to reach with hose streams, especially on account of the dense smoke that prevailed, and would have been quietly handled in the very beginning by two or three heads opening, and it is doubtful if hose streams would have been needed at all.

The "Volturno" disaster is discussed as follows:

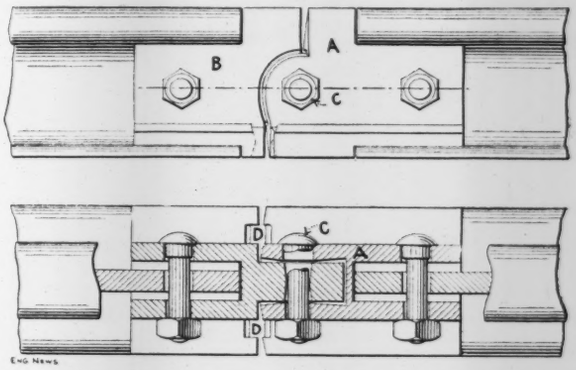
Had the fire on the "Volturno" occurred a few years ago there is no evidence that any aid would have reached them in time, because the position of the nearest vessel as plotted on the chart after all the wireless reports were collected showed that it would have been impossible for them to have seen the burning ship and that all would have been over before any vessel in the ordinary course of events would have seen the smoke from the burning ship. I believe that had the construction been entirely of steel, with sufficient compartments, and above all had sprinkler protection been supplied, the ship would be afloat today.

Whether the fire started from a cigarette or match thrown aside by a steerage passenger, or whether it started from an explosion, or from spontaneous combustion, the automatic sprinkler protection would have taken care of the fire, or, at least, have held it in check until other means could have been used to extinguish it. It would not have traveled through the whole ship, and, even if not completely extinguished, the vessel could have continued to her destination.

Supplementing the argument made in his letter, Capt. Phinney secured a list of steamers equipped with Grinnell sprinklers. This list includes ten ships, as follows: "Alabama," "Western States," and "Detroit the Third," running on the Great Lakes; "Commonwealth," "Plymouth," "Pequannock," "Massachusetts," and "Bunker Hill," in the New England service of the Metropolitan Steamship Line and the New England Navigation Co.; Steam yacht "Cassandra," New York, owned by R. A. Rainey; and steamer "Fort Sutton," San Francisco.

Flexible Rail Joint for Steam-Shovel Tracks

The track on which steam shovels operate needs considerable flexibility to adapt itself to the irregular surface upon which it is laid. The track is in short sections, usually spliced in the usual way, but the splices do not allow for longitudinal bending in a horizontal plane. If the splice joints are bolted tight they do not allow for lateral movement, and there is likely to be breakage or distortion, but if the bolts are left loose enough to allow lateral play, the track may be insecure and lead to derailment, especially as the joint may be covered with mud or water.



Flexible Rail-Joint for Steam-Shovel Tracks; Panama Canal

A special rail joint for this particular service has been used for several months on the Panama Canal, and after satisfactory results with the first lot, a sufficient number was ordered to equip ten steam shovels, each shovel track (in 6-ft. lengths) having 22 joints. The device is shown in the accompanying cut and it will be seen that it consists of two steel castings, bolted to and projecting beyond the rail ends. One casting *A* is of H-section, the end of the web of the rail entering one of the recesses, while the other receives the end of the casting *B*, bolted to the other rail.

Each casting is secured to its rail by a single bolt, while a third bolt *C*, through the engaging portions of the castings, forms a pivot allowing for longitudinal flexibility. The outer or projecting ends of the H-casting are curved laterally, as shown, thus allowing for lateral flexibility. In case this lateral movement should be undesirable at any time, however, it may be prevented by driving spikes in the slots *D*, provided for the purpose.

This joint is the invention of Peter J. Thull, of Culebra, Panama Canal Zone, who was a steam-shovel operator on the canal. Our information is taken from the *Canal Record*.

A Dry-Mixing Machine has been designed specially to meet the requirements of glass works, but is adapted for other plants where fine materials are to be mixed dry. The mixer is of the horizontal-drum type, with or without internal blades, according to the service for which it is required. The revolving drum is entirely inclosed within a dust-proof steel housing, which serves also as the hopper into which the drum discharges the mixed material. The feed spout delivers into the center of the drum, the shaft passing through this spout. The discharge gates are operated through a sleeve mounted on the shaft at the side opposite the feed spout. The drum is operating continuously, being charged and discharged without stopping. At one glass plant a portable mixer is used, the mixer being mounted upon a truck and traveling with its own power under the various overhead bins, from which it is charged through gates. The mixer is supported upon a scale platform, the scale beams being set for the required proportions. The car is run under the first bin, and the material drawn into the drum till the first beam rises. Then the car moves ahead and takes from the second bin, and so on. When the entire charge is secured, the drum starts to revolve and the car is run to the furnace hopper, into which the mixed charge is delivered. This mixer is built by the Stephens-Adamson Mfg. Co., of Aurora, Ill.

Field and Office

A Borough Engineer's Plan-Filing System

By Russell H. Peck*

*Borough Engineer, Palmerton, Penn.

Most engineers and contractors have to keep plans covering various types of work and have some difficulty in finding a serviceable method of not only finding their plans immediately but also keeping track of where they go when they are borrowed, and at what time they are returned. The writer had this same difficulty, but after some puzzling hit upon the following system, which in two years' use has been very efficient, as plans and matter contained in plans can always be found at once and when they are borrowed they are very easily traced.

Filing the Plans

This is just an inexpensive method of filing plans but is elastic, as it can be made up in any finish which the purse permits, and the rest of the system is applicable whether the plans be flat or rolled. The plans are rolled and kept on shelves 5 to 6 ft. wide, 3½ ft. deep, spaced about 6 in. apart vertically. Ordinary string tags of convenient size are attached to a folded corner of the plans by means of an O. K. clip, the tag hanging over the edge of the shelf. Tags should be numbered on both sides, as they tend to twist.

HOME NO. 6101

SUBJECT:-BUILDINGS - CITY

TITLE:-CITY HALL

FOREIGN NO. (For plans from outside)

MADE BY R.H.P. SCALE: 1/4"=1ft

DATE Jan 3 1914 REVISED Jan 16 1914

REC'D:-(For outside drawings)

COPIES: 1-tracing
1original
3Blue Prints

REMARKS See Shelf # 6101

(Typical Card for numerical index)

Fig. 1.

Numbering Drawings

Drawings are numbered according to Divisions, Departments and numbers in the order mentioned. Thus; 6-D-6101 would be drawing number 6101 pertaining to a building belonging to the city. Or, 5-E-5101 would be a water main drawing No. 5101 belonging to the City Water Co. An example of a list of divisions and departments to which these numbers refer might be as follows:

Divisions: 1, Properties; 2, Railroads; 3, Sewers; 4, Drains; 5, Water; 6, Buildings; 7, Streets and Sidewalks; 8, Telephones; 9, Electric Light; 10, Miscellaneous.

Departments: A, General; B, New York Office; C, Local Office; D, City; E, Water Co.; F, Lighting Co.; G, Telephone Co.; H, Disposal Co.; I, Railroad.

These divisions and departments can be varied to conform to almost any kind of office according to local conditions. As to the actual numbers themselves on the plans, they will be numbered consecutively as they are drawn or come in. Each division has all its plans in its respective thousands, and thus allows for a thousand plans. It is further divided into ten subdivisions each allowing for one hundred plans, which can be further subdivided as needed.

Indexing the Drawings

Numerical Index—An ordinary numerical card index is used. Blank tab cards can be ordered and the numbers put on as the index enlarges. The most convenient card size is 4x6 in. A sample of the card used in the numerical index is shown in Fig. 1. The back of the card is ruled for a record of the movements of the drawing; there are columns for date out, number of plans, consignee, return date, and remarks, so that one line covers an entire transaction.

INDEX	TITLE (460 A)	DRAWING NUMBERS
	City Hall, Building Plans	6101
	Cistern, Detail of	5603
	City,- Boundary lines of	1101
	" Terminal Plans	2101
	" Water Wks. Pump Sta. #1 - Details	5316
	(Typical Card under Alphabetical Index)	
	(For Ordinary use without Typewriter)	

Fig. 2.

Alphabetical Index—This is made with the same size cards as the numerical index. Every plan is listed in every possible way. Each item requires only one line on a card. Having found the name of the plan desired by means of the alphabetical index, enter the numerical index with its number, and the proper details will be found on the numbered card and its location at the bottom. In case the plan is not in its place the back of the card will show where it was sent last.

The alphabetical index permits of any degree of cross indexing. For instance, if John Stone, John Jones and Tom Smith all have tracts of land shown on the same plan the index should show the proper plan on which all appear under Jones, Smith, and Stone, all referring to the same plan number. A sample card is shown herewith, Fig. 2.

Any criticisms or suggestions for improvement of the system will be welcomed.

Practical Cost-Keeping System for Contractors

By Everett N. Bryan*

The eternal question with engineer and contractor alike is "What will it cost?" The surest answer lies in data collected from one's own past experience; still it is surprising how widely one must search to find any construction work being executed under a really comprehensive cost system.

The cost-keeping system used by T. K. Beard, contractor, Modesto, Calif., is original and efficient. It is the result of several years' experiment, selection and discard. It accomplishes a twofold purpose. It supplies in detail a record of the cost of the various operations attached to each piece of work. Data thus obtained are used in future estimates, and at the same time supply in excellent detail, and right up to date, information as to the progress of each contract, and the cost of all operations involved, thus making possible a study of men and methods and changes in plans if the work is not progressing satisfactorily.

The means of accomplishing these ends is through a system of weekly reports. The execution of the work involved is under the direction of the engineering department. The head of that department assists in preparing estimates for bids and in planning plant layouts, etc. He is therefore familiar with all details of the contract before the actual work starts, and while the estimate is fresh in mind strikes off a form or outline and forwards it to the timekeeper by which the latter may group all expense items as he reports them in the weekly report.

This form shows to what account each expense item and credit should be charged. It is an analysis of the work to be carried out by the contractor and isolates each individual operation involved. Thus for example, a recent contract comprised work under three main headings:

(1) Steam Shovel Excavation of Shale Rock

(2) Hand Excavation of Hard Basaltic Rock

(3) Hydraulic Excavation of Thin Top Layer of Gravelly Clay.

The work under each of these main headings was further subdivided into the various logical parts representing the more elementary operations. The value of a knowledge of these components is shown by the manner of preparing the bid for the steam-shovel excavation job. The total cost of this work was found by adding the costs of the following minor operations:

1. Plant Delivery
2. Superintendence
3. Breaking Ground
4. Machine Mucking
5. Transportation of Muck
6. Dump Expense
7. Trestle Erection

*Engineer for T. K. Beard, contractor, Modesto, Calif.

WEEKLY REPORT OF LABOR

(Original size of sheet, 8½x14)

Week ending July 12, 1913 **Camp No.** 12

Division	Rate per Day	S	M	T	W	T	F	S	Total Days	Amount	Totals Labor
STEAM-SHOVEL EXCAVATION ACCOUNT (Cont'd.)										$1343.12	
										Pay Roll For'd.	
4. Machine mucking											
A. Operation											
1. Leverman	$6.66	0[5]	1	1	1	1	1	1	6[5]	$43.29	
2. Craneman	5.12	0[5]	1	1	1	1	1	1	6[5]	33.28	
3. Fireman	3.78	0[6]	1	1	1	1	1	1	6[6]	22.68	
4. Pitmen	2.50	0[6]	4	4[6]	5	5	5	3	26[5]	66.25	
5. Shovel rental	15.00	0[6]	1	1	1	1	1	1	6[6]	90.00	
B. Supply expense											
1. Fuel oil			15.75 drums @ $4.00							63.00	
2. Car oil			5.0 gal. @ 25c.							1.25	
3. Waste			4.0 lb. @ 12c.							.48	
C. Repairs and extras											
1. Blacksmith	4.00				1			0[8]	1[8]	7.20	
2. Mechanic	3.50				1	1			2[6]	7.00	
3. B.S. coal			.05 ton @ $26.25							1.31	
4. Hoisting chain										243.36	179.70
5. Transportation											
A. Actual transp.—operation											
1. Dinkey driver	4.75	0[6]	2	2	2	2	2	2	12	57.00	
2. Conductor	2.75	0[6]	2	2	2	2	2	2	12	33.00	
3. Car dumpers	2.50	0[6]	6[5]	7	7	6[5]	7	7	41	102.50	
4. Dinkey rent.	5.00	0[6]	2	2	2	2	2	2	12	60.00	
5. Car rent	.75	0[6]	18	18	20	20	19	17	112	84.00	
6. Track rent	2.50	0[6]	1	1	1	1	1	1	6	15.00	
7. Coal			7.14 tons @ $14.25							101.75	
8. Engine oil			2.0 gal. @ 25c.							.50	
B. Maintenance											
1. Track foreman	3.00	0[6]	1	1	1	1	1	0[5]	5[5]	16.50	
2. Trackmen	2.50	0[6]	2	2	7	5	5	2	23	57.50	
3. Blacksmith	4.00	0[6]				1	1	0[2]	2[2]	8.80	
											275.30
										$1798.12	
										Pay roll for'd.	

WEEKLY COST SHEET

(Original size, 5x8 in.)

Contract No. 9 **Week ending** July 12, 1913

1. Account Steam-shovel excavation

22,855	**Cubic yards progress to date**
$11,252.40	**Total expense to date**
$0.492	**Cost per cubic yard to date**
$0.312	**Cost per cubic yard this week**

2. Account Drilling for steam-shovel excavation

23,432 lin.ft.	**Progress to date**
$1671.35	**Total expense to date**
$0.071 lin.ft.	**Cost to date**
$0.067, lin.ft.	**Cost this week**

3. Account Blasting for steam-shovel excavation

25,940.0	**Cubic yards progress to date**
$1305.99	**Total expense to date**
$0.050	**Cost per cubic yard to date**
$0.046	**Cost per cubic yard this week**

4. Account Trestle erection

43.3 M. ft. b.m.	**Progress to date**
$1526.00	**Total expense to date**
$35.25 M. ft. b.m.	**Cost to date**
Completed	**Cost this week**

5. Account

	Cubic yards progress to date
	Total expense to date
	Cost per cubic yard to date
	Cost per cubic yard this week

SHEET FROM THE GENERAL COST BOOK MADE UP FROM WEEKLY REPORT OF CONTRACT SUPERINTENDENTS

Small upper figures in columns denote costs per cubic yard.

(Original size of book, 12x24 in.)

STEAM-SHOVEL EXCAVATION COST SHEET

Description of work—Canal excavation, 18 ft. base, ¾ on slopes, 70% earth, 30% loose rock **Work for**—Gordon Land Co., by contract No. 9

Report No.	Week ending 1913	Sta. to sta.	Cu. yd.	Plant delivery	Superintendence	Breaking ground	Machine mucking	Transport.	Dump expense	Trestle	Misc. expense	Totals	Remarks
1	May 24			173.12								173.12	
2	May 31			362.35								362.35	
3	June 7			495.13	47.73	67.37				136.10	*346.17	1091.90	
				120	078	290	267	170	111	1361	*593	2990	
4	June 14	216+50—217+20	563	67.20	43.86	162.60	149.71	95.60	62.77	767.47	*333.28	1,682.49	*Establishing camp and water supply
16	Sept. 6			596.15	12.70				14.70	13.18	*115.76	752.49	*Camp demolition
17	Sept. 13			392.67								392.67	
18	Sept. 20			225.45								225.45	
			55,348	2399.73	844.28	5613.33	4956.28	5369.81	1033.36	1684.18	913.50	22,814.47	Grand totals
				043	015	101	090	097	019	031	017	413	Cost per cu.yd.

BLANKS USED IN A CONTRACTOR'S COSTS-KEEPING SYSTEM

(Black-face type indicates printed matter.)

8. Riprapping Sides of Embankment Made with Shovel Muck

9. Deepening 18 in. the Canal from Whose Sides Shovel Was Excavating.

10. Miscellaneous Expense.

This subdivision is very different from the conventional analysis of costs whose subdivisions are "Labor, Machine Rental, Fuel, etc." The present method makes each of the elementary operations involved an account by itself, and the data for it are complete. It would be difficult to imagine two pieces of steam-shovel work so dissimilar that the data from some of the divisions of one could not be used in estimating upon the other. In very hard rock the cost for "breaking ground" will run high, and in all probability the cost of other operations will increase slightly, due to the decreased yardage excavated per shift. But a small experience should teach anyone quickly the possible yardage for any particular size shovel in the various classes of material, if the shovel is free to work out its capacity without being limited by other operations involved.

In regard to "Riprapping" and "Deepening Canal": the former covers the expense involved in riprapping with the larger rock from the shovel muck, the slopes of the fill into which the muck was wasted; the latter covers a portion of the expense involved in lowering some 12 to 18 in. the old bottom of the canal from the side bank of which the shovel was excavating. This material had to be drilled, blasted, and finally mucked by the shovel, and all these operations were properly charged to the ordinary operations of "breaking ground" and "machine mucking." But about one-half the total of this bottom yardage had to be hand mucked and piled up on the center line of the old canal section to allow the passage of trains and to act as a wall in preventing slides from covering the track when the bank was blasted. This hand mucking was unusual and in no way productive of progress; it was therefore deemed advisable to create a subdivision for it, which would make it possible to always eliminate that expense from the data in figuring new work.

That idea is characteristic of the whole scheme of analysis and subdivision of costs, i.e., to so separate and distinguish each minor operation involved in a piece of work as to make the data upon it complete. Everyone knows how infrequently one finds a second piece of rock work, dirt work, concreting, or even bridge work which is exactly like an earlier job. This argument is often used against the keeping of cost data at all. But with records segregated as outlined above, there need be no serious handicap in estimating. One can go to two or three (or more) previous contracts and pick from each rock work, dirt work, concreting, or even bridge work For instance, data on minor operations, such as drilling, shooting or transportation, might be borrowed from the steam-shovel records and applied to work as dissimilar as car and trap or team work.

But the scheme of subdivision does not stop at naming and making complete the data for the various minor operations. These are again subdivided into more elementary operations where such subdivisions may seem useful. Thus the subdivision of the main minor operations above named would give the total cost and the cost per cubic yard of erecting the trestle from which the steam-shovel muck was deposited in the fill. But this would be of little value in checking up the efficiency of the carpenter force in erecting the trestle or in estimating the cost of another trestle. To be useful the records must show cost in terms of thousand feet board measure of lumber placed. Trestle erection expense items are therefore kept in a separate account besides being included in the steam-shovel excavation account, and the following subdivision of items made:

7. Trestle Erection
   (A) Mucking for Foundations
   (B) Carpentry
   (C) Materials

and the account shows costs in terms of thousand feet board measure placed.

Similarly in the matter of drilling, it is not sufficient to know costs in terms of cubic yards drilled as would be shown in the steam-shovel excavation account. One foot of hole may break a fraction of a cubic yard or many cubic yards and drilling costs vary as the length of the hole drilled. To be useful, therefore, drilling records must be in terms of lineal feet of hole drilled and, accordingly, a separate individual account is run for drilling outside the steam-shovel excavation account—one with its own subdivisions and showing costs in terms of lineal feet of hole.

Also in many other operations, e.g., "Transportation," the policy is pursued of encouraging the grouping of items on the report. Transportation is divided into operation expense and maintenance expense. No additional effort attaches to making such segregations and it accomplishes the twofold purpose of teaching the timekeeper to scrutinize and classify each item of expense closely, and makes the report read so plainly of the work as it progresses that frequent trips to the ground itself for information are unnecessary.

Besides the various income accounts there will always be connected with a piece of work certain non-income accounts, by which are meant accounts taking care of expense items which are not directly productive of progress. The following four non-income accounts are generally absolutely necessary.

(1) A Kitchen Account, to which is charged the cook and his helpers' expense and the supplies, and which should balance by credits from meals furnished.

(2) A Stock Account, to which is charged corral-boss, hay, grain, shoeing and other stock expense, and which should balance by credits from stock rental.

(3) A General Hauling Account, to which is charged the hauling of commissary and camp supplies and stock supplies, e.g., cement, steel, lumber, etc., and which should balance by credits from proper additional charges made for the materials when actually used.

(4) A Camp Equipment Account, to which is charged the expense of erecting and maintaining the camp, its water supply, etc., and which must remain unbalanced unless credited at the end of the job with the accommodation of the maximum number of men cared for by it, or unless prorated and charged by transfer to the miscellaneous expense item of the various income accounts.

Experience has taught so far that it is best not to attempt to make complete reports of the non-income accounts on the weekly reports but to insist on the timekeeper running separate accounts for each at the camp. It is not desired to confuse the weekly reports, which are mere summaries, with a mass of commissary detail. Accordingly, for the non-income accounts only the state-

items of labor expense are included; and in the matter of the income accounts, while the materials expense is also included, only the labor expense is extended forward to the totals column. This makes it possible to check up the monthly pay roll by merely adding up the extensions made for labor into the totals columns of the weekly reports.

Each contract superintendent is supplied with the necessary engineering assistance to set grade and alignment stakes, assist in plant layouts, and to report weekly progress for all departments of the work. This progress is incorporated in the weekly report before mailing to the central office.

At the central office these camp weeklies pass first through the hands of the chief engineer for a cursory examination to determine if proper classifications have been observed and to note any queries or remarks, for questions concerning charges and credits often arise. They are then turned over to a clerk who works them up and enters them into the "cost book." In this "cost book" there is a separate sheet for each account, and it is so ruled that the data of each week are placed on a separate line, with the figures for date, report number, progress, and expense for the various operations, each in a column by itself with proper heading. Then there is a final total column in which is entered the combined cost of all the operations. These figures, showing in the proper column the cost for each operation and in the final column the combined cost of all operations, are made in black ink, and over them in small red figures is set down the cost per cubic yard, or other unit cost as the case may be. (See the sample cost sheet.)

The clerk also makes out a "Weekly Cost Summary" which shows for each of the accounts the total expense to date, the total progress to date, the percentage completed, the average unit cost to date, and the unit cost of the work done the preceding week. This summary is made on a regular printed form in triplicate and one copy turned over to the contractor himself, one to the general superintendent, and a third forwarded to the contract superintendent.

The effort connected with this cost-data work is not so tedious or so expensive as one might suppose. On the smaller contracts the superintendent is his own timekeeper and makes out his own reports. On the slightly larger contracts the superintendent is furnished with a timekeeper, who usually has had some slight engineering experience and who will be able to measure and report progress. On the largest contracts, besides the timekeeper, each contract, or if conveniently located each group of contracts, is furnished with an instrument-man competent to set grades, cross-section, or alignment stakes, measure and report progress, and do any one of the many little things which require some engineering experience and which if left undone will cost in a month's time much more than the instrument-man's salary.

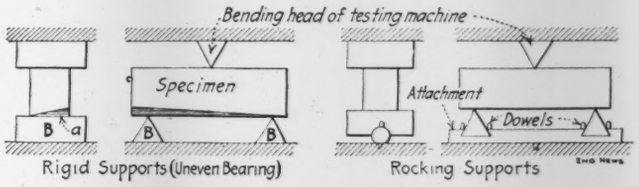
On the larger concrete work an inspector is sometimes placed on that particular part of the work. He takes batch measurements of the concrete poured, reports daily the cement, sand, gravel, lumber, steel and other supplies and the labor used, so that each night the superintendent, who may have spent only a part or none of the day on that particular part of the work, knows exactly what his day's pour has cost. The advantages of such knowledge is not imaginary. In several apparently trivial matters many months' salary have been saved. The reports, when many contracts are in progress, require a considerable portion of one man's time for supervision only, a man who can be on the ground and in the office and see to it that proper classifications are made of all expense items and none overlooked. In the office, one clerk will have no difficulty in taking care of the reports from eight or ten contracts and doing much other work besides, such as checking pay rolls, etc.

The exact value of the cost system would be difficult to determine but it is nevertheless real. To know at all times just what every portion of the work is costing acts as a whip all along the line. A low mark in cost set one week encourages the contract superintendent to hold it there continuously, or if set by another contract superintendent creates a spirit of healthy rivalry. If costs are running high, additional efforts are made to reduce them. The general superintendent lends his aid and the problem is attacked from every angle until either a reduction in cost is accomplished or a revision of ideas thereon.

Rocker Support for Lower Bearings in Transverse Tests

By Julian C. Smallwood*

When making transverse tests on specimens of rectangular section, difficulty is encountered if the lines of support on the two lower bearings are not parallel. Such a condition, which results from non-parallelism or warped shape of the surfaces of the test-piece, necessitates shimming, as at *a*, between the bottom surface of the specimen and one or both of the bending dies *B B*, to get a fair bearing. This is especially troublesome with brittle materials.



Having had occasion to test a large number of concrete building blocks, the writer devised the attachment shown by sketch. This not only adjusts the specimen so that it bears nicely on both bending dies and is uniformly loaded, but it spaces the specimen so that the load is applied at the center of the required span.

The attachment consists of a 1¼-in. bar planed along its length so as to present a flat surface 1 in. wide. At each end, opposite the flat, there is a pair of ¼-in. dowels, beneath which a saddle is thus formed upon which a bending die may rock. The dies were clamped together with their bottom surfaces in contact, and a 1¼-in. hole drilled through them so as to make a semicircular slot in each. These slots fit the saddles, so that when the dies are placed on the attachment, as shown in the sketch, they will assume any position necessary to make full contact with the bottom surface of a specimen of irregular shape.

The dowel on the under side of the attachment fits into a hole drilled in the center of the platform of the testing machine, thus centering the dies.

The saddle dowels may be fitted to a number of holes in the top of the attachment to provide for various spans.

*617 Walnut Ave., Syracuse, N. Y.

Erecting a Heavy Warehouse with Steel Columns and Girderless Concrete Floors

By Peter B. Sheridan*

The largest building erected in Boston during the last year has just been completed ready for occupancy. This is the 10-story, 100x271-ft. building of the Paine Furniture Co., St. James Ave. and Arlington St. The architects are Densmore & Leclear, Boston, and the contractors are James Stewart & Co., Inc., New York. The building has steel columns fireproofed with concrete, and reinforced-concrete girderless slab floors, except that the interior columns above the 8th floor are 2-ft. round reinforced-concrete columns. The exterior columns are steel (Bethlehem H-section) from footing to roof level. Shelf and stiffener angles on the columns at the base of the column cap support the floor.

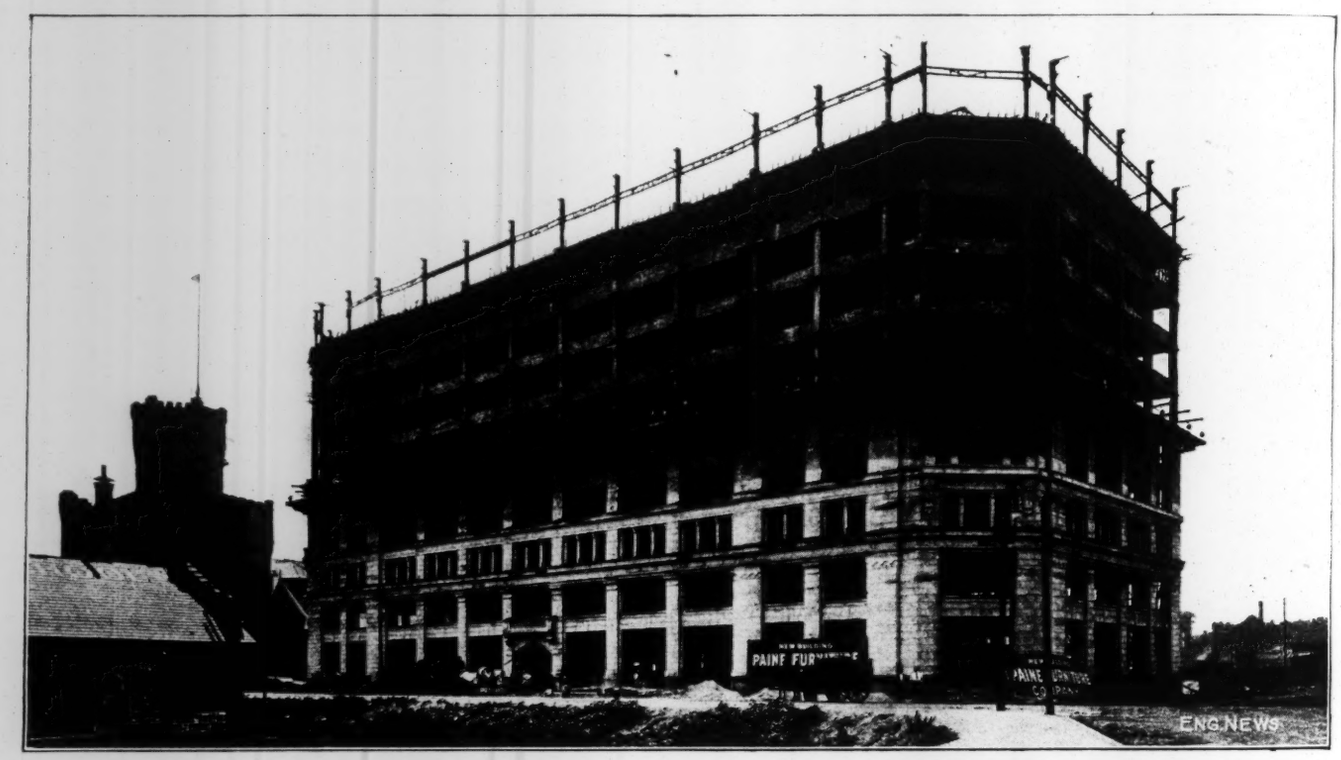


Fig. 1. Paine Furniture Building, Boston, Ready for Erecting Top Story (Sept. 29, 1913).

The bays are about 20 ft. each way. The columns are connected in the outer bay by 1-in. tie-rods at each floor. The floors are 8-in. slabs with multiple-way reinforcement, the rods being bent up near the columns to give shear reinforcement. The footings are mainly pile footings supporting independent piers, but a group of 11 columns (out of the total of 91) rests on a reinforced-concrete slab or mat supported by some 900 piles, 30 in. on centers.

The contractors began work Jan. 4, 1913. A No. 0 Thew steam-shovel handled the 13-ft. cellar excavation except at the deeper boiler room. The dirt was teamed out over plank runways. Much water was struck, some of it from an underground stream, and at times the pulsometers were kept going 24 hr. The boiler-room excavation was taken out by derrick-bucket.

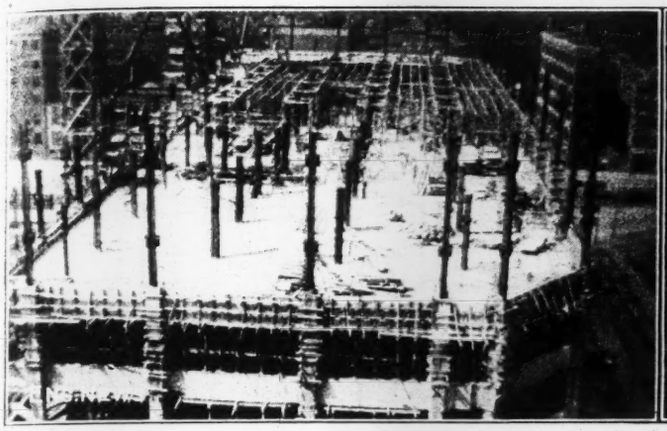
Piledriving was sublet to James Scully & Co., who had three drivers on the work. About 5000 spruce piles were driven (with bark on), up to 30 ft. deep, through mud and sand to hard clay. Each pile was figured safe for 10 tons; the last three penetrations (2400-lb. hammer, 15-drop) averaged 2½ in. In a test of a 25-ft. pile the day after driving, two blows of 15 ft. gave no penetration whatever.

While excavation and piledriving was in progress a central boiler plant was installed about 50 ft. from the rear of the building to supply steam for derricks, hoists and mixers, and a small sawmill and a blacksmith's shop were equipped. Two steel derricks with 95-ft. mast and 85-ft. boom were set in the foundation pit. Outside and opposite the derricks two concrete mixers were set up.

Placing the footing forms began as soon as some of the pile foundations were ready and cut off. The footing forms were made so that they could later be used as sides of spandrel beams, which were of about the same depth as the footings; the material of the basement wall forms was similarly used for the spandrel beams. The footings, basement floor girders (under walls) and floor slab (4 in., under the waterproofing) were all concreted by the derricks, using ¾-yd. buckets. Footings were stripped a day after pouring. The mat footing was poured in sections, the reinforcement, however, being continuous.

The entire basement floor was waterproofed with tar paper and tar pitch over the footings and the bottom slab of the floor. Where the column bases were to be set, the waterproofing was protected by a copper plate 4 ft. square, over which another layer of waterproofing was placed. The whole surface of the waterproofing was covered with

*Construction Engineer, Hazleton, Penn.



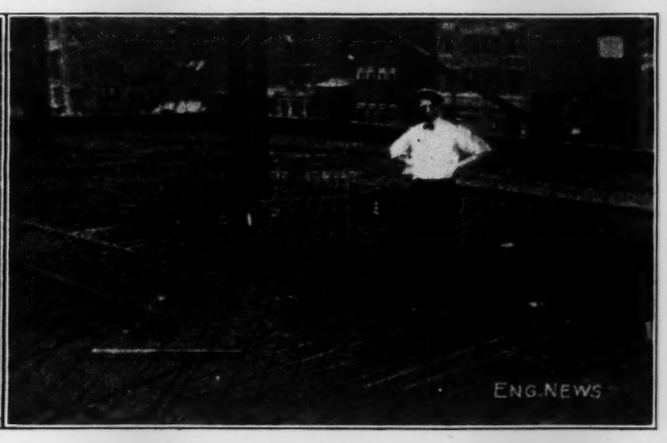


Fig. 4. Slab Reinforcement Ready for Concrete

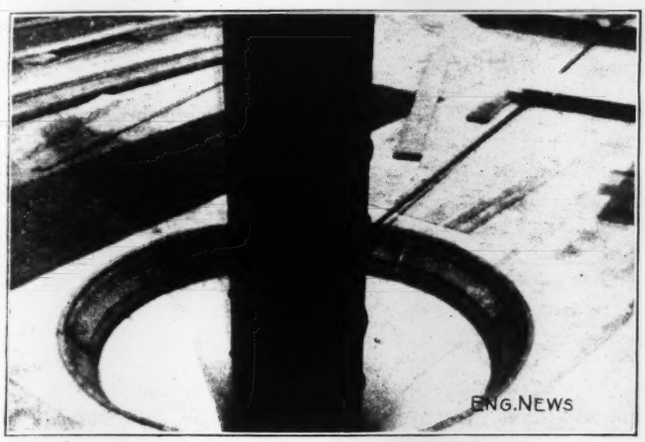


Fig. 2. Progressive Floor Work
Fig. 3. Column-Cap Mold and Floor Forms Ready to Place Slab Reinforcement

FIGS. 2-4. THREE VIEWS OF FLOOR WORK IN PAINE FURNITURE BUILDING

1 in. of 1:2 grout, on which the column bases were set (cast-iron for exterior, 6-in. steel slabs for interior columns).

The concreting derricks set the column bases and the lower sections of the columns. The lower sections of the interior columns were two stories high, while the exterior columns extended only to the first floor. Above this, however, all columns were in two-story lengths, thus breaking joints. All columns were wrapped with ⅜-in. spiral spaced 12 in., tied to four ¾-in. longitudinal rods, as bond for the concrete fireproofing.

A set of forms for a complete floor together with shoring frames (bucks) had already been made, ready to place. The work of laying the forms was carried on in a systematic way; the bucks and column-head forms (8 ft. square, made in two pieces to facilitate placing around columns, and with a 6-ft. hole for the cap form) were placed in position by a gang of carpenters four or five days in advance of the gang placing the 4x10-ft. floor panels and the spandrel-beam and exterior-column forms. The forms for incasing the interior columns were galvanized iron 2 ft. diameter and were placed as the floor-form work progressed. A galvanized cap form was placed in the 6-ft. hole from above, this form tapering down to the column form.

The same forms and bucks were used from the first floor to the roof with but little patching, great care being taken in the stripping and handling. The forms when loosened from the concrete were dropped on ropes a few feet from the ceiling and lowered to the floor, from where they were hoisted through the elevator openings by Chicago booms on the concrete hoists to the floor above. The reinforcing steel was also hoisted by the Chicago booms. The rods in the diagonal band, which were bent up near the cap, were laid straight and later bent up by a double hickey, a two-prong bending lever, this being a quicker way than to bend them on the bench. The two inner rods of each band were bent halfway around the columns and continued on 6½ ft. beyond.

The columns were filled at least 6 hr. before concreting the floor, to give time for shrinkage. All concrete was placed with Ransome buggies, as spouting or chuting of concrete was not allowed. Two concrete hoists were used, each averaging a bucket a minute. When a floor was just starting, runways were laid on the bucks to the farther hoist. Floors were stripped about 48 hr. after concreting, and the floor then properly shored, the shoring being left in place two weeks.

All concrete was 1:2:4 mixture, using Dragon cement, Plum Island sand, and trap rock.

Good speed was attained in the floor work toward the end. The last four floors were done in 32 days.

The carpenters placing form work were kept at least three days in advance of the reinforcing-steel men, and they in turn kept three days in advance of the concrete gang.

At floors where the interior columns were connected, the columns were placed before any bucks were set, but the exterior columns were generally placed after. All the columns were hoisted and placed by breast derricks. They were plumbed and kept plumb by turnbuckles and wood trusses (see Fig. 1) till after concreting. Particular care was taken with the exterior columns to prevent them from leaning out, as it was a very easy matter for a column to lean out two or three inches at the top, and should this occur great trouble would be encountered with the face brick and terra cotta, which, of course, must be carried up plumb.

The basement top floor (8 in.) was laid after the construction had reached about the sixth floor, this work and almost all interior brickwork being held for rainy days. The setting of limestone was commenced after the fourth floor was reached, the masons at first working on a pole scaffold. When the sixth floor was reached in the construction, a "Perfect" safety scaffold was placed, this being suspended from I-beams extending from the sixth

floor over the spandrel beams and fastened on the inside by U-bolts extending through the concrete floor and over the I-beams. By the time the tenth floor was reached in the construction, the terra cotta and face brick had been set to the sixth floor; the I-beams were then extended from the tenth floor. When the masons reached this point, a wood scaffold was built on top of the I-beams to finish the terra cotta cornice.

All material for masons was hoisted by two brick hoists placed in the passenger-elevator shaft in the center of the building. The required terra cotta for spandrel beams and brick for piers was always hoisted and placed on the two floors above where the masons were working, so that when the masons advanced to a floor they never had to wait for material.

The terra cotta for spandrel beams was supported by an angle fastened to bolts at the bottom of the beam, these bolts and also anchors for the terra cotta having been placed in the forms before concreting. Angles fastened to columns supported the face brick from floor to floor.

The structural part of the building was completed in the fall, 1913, and by Dec. 15 the building was practically finished except for some interior-equipment items.

Approximate Long Chords

In fieldwork it is necessary to compute long chords where no trigonometrical tables are available, and thus delay is caused, or else a series of smaller chords must be used, increasing the liability of error both as to distance and to line.

The following short-cut, while approximate, is certainly of sufficient exactness for all kinds of fieldwork.

Let

S = length of arc to be measured;
R = radius of curve;
C = long chord;
a = angle subtended by arc S, in circular measure.

Then

$$a = \frac{S}{R}, \text{ and}$$

$$C = 2\,R\,sin\,\frac{S}{2\,R}$$

The expansion of the sine may be put

$$sin\,a = a - \frac{a^3}{3\,!} + \frac{a^5}{5\,!}$$

a rapidly converging series for values of a less than unity (a expressed in circular measure). Hence,

$$C = 2\,R\left[\frac{S}{2\,R} - \frac{1}{3\,!}\left(\frac{S}{2\,R}\right)^3\right]$$

the error involved in omitting the third term being

$$2\,R\left[\frac{1}{5\,!}\left(\frac{S}{2\,R}\right)^5\right]$$

Rewriting the expression for C, we obtain

$$C = S - \frac{S^3}{24\,R^2}$$

or the chord-length is found by subtracting from the arc length the quantity

$$\frac{S^3}{24\,R^2}$$

The error in the approximation to the third place may be estimated by taking the extreme case of $S = R$, when it is

$$\frac{R}{1920}$$

Table I, herewith, gives the corrections to be applied to the arc lengths to get the chords, for radii from 150 to 2000 ft., and arcs from 25 to 120 ft.

In railroad work, S is usually expressed as so many stations and a plus. To get the S of the formula multiply the length in stations by 100

Let

n = length of arc expressed in stationing;
D = degree of curve.

Then, since D is the central angle corresponding to a 100-ft. arc, the correction (difference between arc and chord) becomes:

$$\frac{n^3\,D^2}{24}$$

if D is expressed in radians. If D is given in degrees the correction becomes

$$\frac{n^3\,D^2}{787}.$$

the result being expressed in feet.

TABLE I. CORRECTIONS FOR CHORDS, WHEN ARCS ARE GIVEN IN FEET

	S										
R	25	30	40	50	60	70	80	90	100	110	120
150	0.03	0.05	0.12	0.23	0.40	0.63	0.95	1.35	1.85	2.46	3.18
175	0.02	0.04	0.09	0.17	0.29	0.47	0.70	0.99	1.36	1.81	2.35
200	0.02	0.03	0.07	0.13	0.23	0.36	0.53	0.76	1.04	1.39	1.80
225	0.01	0.02	0.05	0.10	0.18	0.28	0.42	0.60	0.82	1.09	1.42
250	0.01	0.02	0.04	0.08	0.14	0.21	0.34	0.48	0.67	0.89	1.15
275	0.01	0.01	0.04	0.07	0.12	0.19	0.28	0.40	0.55	0.73	0.95
300	0.01	0.01	0.03	0.06	0.10	0.16	0.24	0.34	0.46	0.61	0.80
325	0.01	0.01	0.03	0.05	0.09	0.14	0.20	0.29	0.39	0.52	0.68
350	0.01	0.01	0.02	0.04	0.07	0.12	0.17	0.25	0.34	0.45	0.59
375		0.01	0.02	0.04	0.06	0.10	0.15	0.22	0.30	0.39	0.51
400		0.01	0.02	0.03	0.06	0.09	0.13	0.19	0.26	0.35	0.45
425		0.01	0.01	0.03	0.05	0.08	0.12	0.17	0.23	0.31	0.40
450		0.01	0.01	0.03	0.05	0.07	0.11	0.15	0.21	0.27	0.36
475		0.01	0.01	0.02	0.04	0.06	0.09	0.13	0.18	0.25	0.32
500		0.01	0.01	0.02	0.04	0.06	0.09	0.12	0.17	0.22	0.29
550			0.01	0.02	0.03	0.05	0.07	0.10	0.14	0.18	0.24
600			0.01	0.01	0.03	0.04	0.06	0.08	0.12	0.15	0.20
650			0.01	0.01	0.02	0.03	0.05	0.07	0.10	0.13	0.17
700			0.01	0.01	0.02	0.03	0.04	0.06	0.09	0.11	0.15
750			0.01	0.01	0.01	0.03	0.04	0.05	0.07	0.10	0.13
800				0.01	0.01	0.02	0.04	0.05	0.07	0.09	0.11
850				0.01	0.01	0.02	0.03	0.04	0.06	0.08	0.10
950				0.01	0.01	0.02	0.02	0.04	0.05	0.06	0.09
1000				0.01	0.01	0.01	0.02	0.03	0.05	0.07	0.08
1500						0.01	0.01	0.01	0.02	0.03	0.03
2000							0.01	0.01	0.01	0.01	0.02

R = Radius of curve.
S = Arc length.
Tabular value subtracted from arc length gives chord, in ft.

TABLE II. CORRECTIONS FOR CHORDS, WHEN ARCS ARE GIVEN IN 100-FT. UNITS

		n								
D	R	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
2.5	2292	0.008	0.03	0.06	0.12	0.22	0.34	0.51	0.72	0.99
2.4	2390	0.007	0.02	0.06	0.11	0.20	0.31	0.47	0.67	0.92
2.3	2494	0.007	0.02	0.05	0.11	0.18	0.29	0.43	0.61	0.84
2.2	2607	0.006	0.02	0.05	0.10	0.17	0.26	0.39	0.56	0.77
2.1	2730	0.006	0.02	0.04	0.09	0.15	0.24	0.36	0.51	0.70
2.0	2865	0.005	0.02	0.04	0.08	0.14	0.22	0.33	0.46	0.64
1.8	3185	0.004	0.01	0.03	0.06	0.11	0.18	0.26	0.38	0.51
1.6	3580	0.003	0.01	0.03	0.05	0.09	0.14	0.21	0.30	0.41
1.4	4110	0.002	0.01	0.02	0.04	0.07	0.11	0.16	0.23	0.31
1.2	4780	0.002	0.01	0.01	0.03	0.05	0.08	0.12	0.17	0.23
1.0	5730	0.001		0.01	0.02	0.03	0.05	0.08	0.12	0.16
0.8	7150	0.001		0.01	0.01	0.02	0.03	0.05	0.07	0.11
0.6	9550	0.000			0.01	0.01	0.02	0.03	0.04	0.06

D = Degree of curve.
R = Radius.
n = number of 100-ft. stations in arc.
Tabular value subtracted from arc length in ft. gives chord, in ft.

Table II gives the corrections for flat curves with long chords, according to this equation.

The corrections of Table II (as well as those of Table I) are to be applied to the true *arc* length, which is the basis of measurement when laying out the fieldwork from office-drawn plans. However, where the 100-ft. chord is used as a unit of arc-length, the corrections will be slightly less. Obviously, the correction for $n = 1$ should be zero. For any other value of n, reduce the figure given in Table II as follows: Take the tabular figure for $n = 1$, corresponding to the D or R of the particular case, and multiply it by the actual n; subtract the product from the tabular value given in the proper n column. To facilitate making this reduction, the correction in the "$n = 1$" column is given to the third place.

The third term in the expansion is

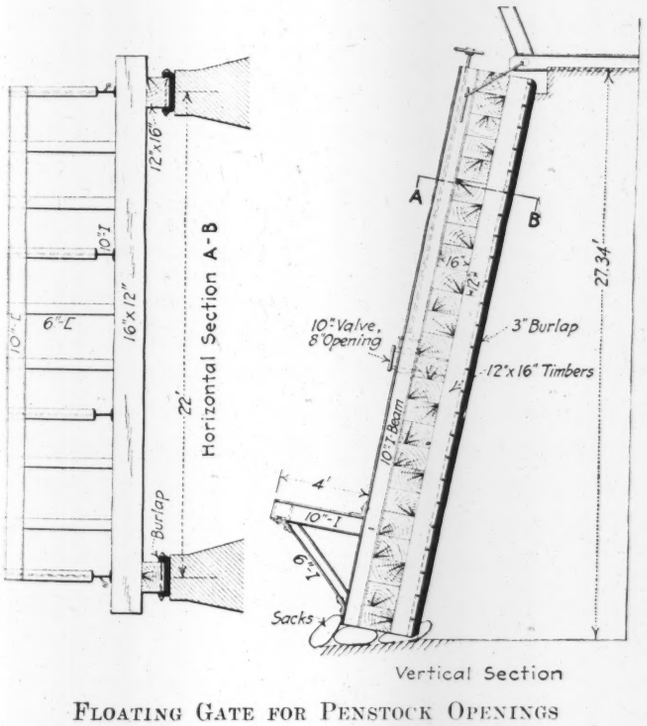
$$\frac{R}{1920}\left(\frac{S^5}{R^5}\right)$$

If this is equated to 0.01 ft (as close as the table is computed) we get $S = 1.8\ R^{\frac{4}{5}}$. For values of S greater than this a correction must be applied. On applying this to Tables I and II, it will be found that only the four upper right-hand figures of Table I are affected; i.e., except for the values for $R = 150$ to 175, $S = 110$ to 120, all values in both tables are correct to 0.01 ft.

The method given in this article is time-saving even where trigonometric functions are handy, since the correction may be obtained in a single process, by slide-rule, as against the multiple process when tables are used.

Floating Gate for Penstock Openings

A floating gate or bulkhead for use in a hydraulic power plant is shown in the drawing herewith, from F. H. Barrett, Parr, S. C. It is a solid timber gate pad, 16 in. thick, tied and stiffened by four 10-in. I-beams on the back and two jamb-pieces 12x16 on the face. The latter are faced with burlap to a thickness of 3 in., which pads make tight contact with the wall or pier around the gate opening, when the gate is in use. A bottom seal of cement bags or the like may be used. To assist in handling the gate, a shelf or rack is built out on the outer face, near the foot. Ballast placed on this rack sinks the gate and holds it down when in place.



FLOATING GATE FOR PENSTOCK OPENINGS

A pipe with 10-in. valve passes through the gate near midheight. This valve would be open while the gate is being adjusted.

Attachment for lateral stays is provided near top and bottom of the two outer I-beams, giving a means for holding the gate securely in place.

The gate worked well in 30 ft. of water. It is a handy thing to have around any hydraulic plant, especially where there is no provision for stoplogs.

Lowering Groundwater by Pumping, to Lay Pipe in Quicksand

In the construction of a recent addition to the water-supply of Battle Creek, Mich., pipes were laid in water-bearing quicksand by the aid of adjacent drainage wells driven below the bottom of the pipe trench which was then kept comparatively dry by pumping from the wells. The method was described by W. W. Brigden, Engineer and Superintendent of the Department of Public Works, of the City of Battle Creek, before the annual meeting of the Michigan Engineering Society in January, 1914.

The pipes were 15 and 18-in. diameter and were laid in trenches from 3 to 4 ft. wide and from 12 to 18 ft. deep through an extremely water-bearing quicksand. The pipe line was part of a water-supply system leading water from artesian wells to a basin underneath the pumping station. The trenches were sheeted with 2x10-in. planks 14 ft. long, and 3x8-in. rangers, and where the depth went below 14 ft., 1-in. triple-lap Wakefield piling was used. Quicksand was found from 1 to 3 ft. below the surface, and in most cases extended to the bottom of the trench. In the places where it did not reach the bottom of the trench a sandstone rock was found. In this formation it was quite easy to keep the trench dry, but where only boulders were found near the bottom, or where the sand formation continued clear to the bottom, it was practically impossible by ordinary pumping methods to keep the water from the trench sufficiently to allow the laying of the pipe. Not only did the water come in to fill up the trench, but the quicksand ran into the sides and displaced the pipe which had just been laid.

In this part of the trench the ordinary system of trenching and pipe-laying was abandoned, and the drainage-well system installed. In this latter construction 2-in. well pipes were driven from 10 to 20 ft. apart along one side and parallel to the trench. These pipes were first driven only a foot or so below the bottom of the trench, but it was discovered that this was not deep enough to keep the trench dry and the well point had to be driven so that its lower end would be at least 5 ft. below the trench. The various wells were connected up by means of pipes and were tied into a steam pump which was kept working 24 hr. a day.

The line of wells was laid out on one side of the trench so as to not interfere with the excavation, which was thrown to the other side, and the wells were placed 4 or 5 ft. outside the sheeting. This was found to be too close to the trench, for when the ditch was dug nearly to its bottom the quicksand between the well point and the bottom of the trench became the line of least resistance and air passed through from the trench to the point, thus greatly lessening the amount of the water that could be removed and on a few occasions letting in so much air that the water could not be raised the necessary 15 ft. until the pumps were placed in the very best condition and some of the poorer wells disconnected from the system. A few of the well points were split by striking boulders. Some of these were removed at the time of driving, but in the case of others the injury was not discovered and these wells were in poor condition to screen the fine sand, and a considerable quantity passed through the water. The sand at times filled the pipes in the wells to such an extent as to stop pumping.

*

Table Giving Voids in Sand, Based on Weight Method

By S. Whinery*

The old method of determining the voids in sand by filling a measured volume of sand with water in cumbersome, and unless done with care, is likely to be inaccurate, because of the entrained air that is difficult to entirely avoid.

The method described herein, which is much better and simpler, was used more than 20 years ago by the writer, and the table offered herewith was prepared to facilitate and expedite the work. The method is based upon the fact that most sand is practically all silica or minerals of about the same specific gravity as silica, that is, 2.66. It consists in taking the weight, in grams, of 100 c.c. of the dried sand and computing the voids by the formula:

$$\textit{percentage of voids} = \frac{266 - \textit{wt. of 100 c.c. of sand}}{2.66}$$

The table was computed by this formula.

The apparatus required is a 100 c.c. flask and a balance reading to tenths of grams, or even to the nearest gram, which is sufficiently close for practical purposes. (It will be noticed that the difference in percentage of voids for a whole gram, as for instance 180 grams and 181 grams is only about one-third of one per cent.)

A small admixture in the sand of minerals other than silica, will not greatly change the accuracy of the result. Thus, if the sand contains as much as 20% of limestone, granite, or serpentine, the table will still give results correct to within a small fraction of one per cent. of voids. As in any other method of determining voids, the degree to which the material is packed by shaking down will affect the result materially.

*Consulting Engineer, 95 Liberty St., New York City.

TABLE FOR THE DETERMINATION OF THE PERCENTAGE OF VOIDS IN SAND

Assuming the specific gravity of silica = 2.66

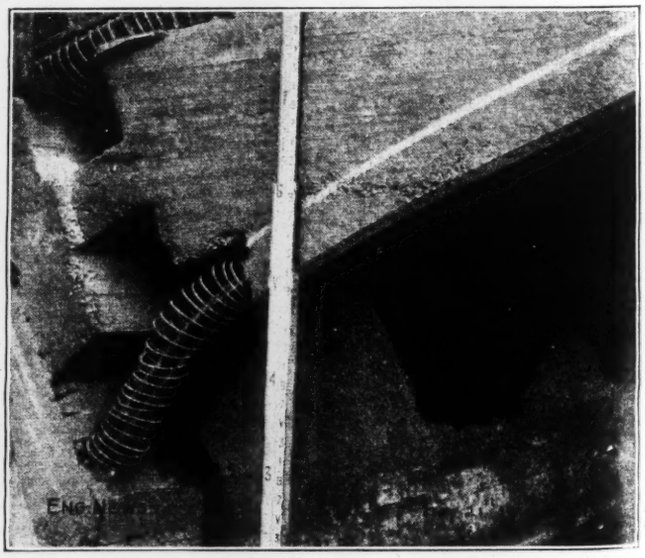
Weight of 100 c.c. sand	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
160	39.85	39.81	39.79	39.74	39.70	39.66	39.62	39.58	39.55	39.51
1	39.47	39.43	39.39	39.36	39.32	39.28	39.24	39.20	39.17	39.13
2	39.09	39.05	39.02	38.98	38.94	38.90	38.87	38.83	38.79	38.76
3	38.72	38.68	38.64	38.61	38.57	38.53	38.49	38.45	38.42	38.38
4	38.34	38.30	38.27	38.23	38.19	38.15	38.12	38.08	38.04	38.01
5	37.97	37.93	37.89	37.86	37.82	37.78	37.74	37.70	37.67	37.63
6	37.59	37.55	37.51	37.48	37.44	37.40	37.36	37.32	37.29	37.25
7	37.21	37.17	37.14	37.10	37.06	37.02	36.99	36.95	36.91	36.88
8	36.84	36.80	36.76	36.73	36.69	36.65	36.61	36.57	36.54	36.50
9	36.46	36.42	36.39	36.35	36.31	36.27	36.24	36.20	36.16	36.13
170	36.09	36.05	36.01	35.98	35.94	35.90	35.86	35.82	35.79	35.75
1	35.71	35.67	35.64	35.60	35.56	35.52	35.49	35.45	35.41	35.38
2	35.34	35.30	35.26	35.23	35.19	35.15	35.11	35.07	35.04	35.00
3	34.96	34.92	34.89	34.85	34.81	34.77	34.74	34.70	34.66	34.63
4	34.59	34.55	34.51	34.48	34.44	34.40	34.36	34.32	34.29	34.25
5	34.21	34.17	34.13	34.10	34.06	34.02	33.98	33.94	33.91	33.87
6	33.83	33.79	33.76	33.72	33.68	33.64	33.61	33.57	33.54	33.50
7	33.46	33.42	33.38	33.35	33.31	33.27	33.23	33.19	33.16	33.12
8	33.08	33.04	33.00	32.97	32.93	32.89	32.85	32.81	32.78	32.74
9	32.70	32.66	32.63	32.59	32.55	32.51	32.48	32.44	32.40	32.37
180	32.33	32.29	32.25	32.22	32.18	32.14	32.10	32.06	32.03	31.99
1	31.95	31.91	31.88	31.84	31.80	31.76	31.73	31.69	31.65	31.62
2	31.58	31.54	31.50	31.46	31.43	31.39	31.35	31.31	31.28	31.24
3	31.20	31.16	31.13	31.09	31.05	31.01	30.98	30.94	30.90	30.87
4	30.83	30.79	30.75	30.72	30.68	30.64	30.60	30.56	30.53	30.49
5	30.45	30.41	30.38	30.34	30.30	30.26	30.23	30.19	30.15	30.12
6	30.08	30.04	30.00	29.97	29.93	29.89	29.85	29.81	29.78	29.74
7	29.70	29.66	29.62	29.59	29.55	29.51	29.47	29.43	29.40	29.36
8	29.32	29.28	29.24	29.21	29.17	29.13	29.09	29.05	29.02	28.98
9	28.94	28.90	28.87	28.83	28.79	28.75	28.72	28.68	28.64	28.61
190	28.57	28.53	28.49	28.46	28.42	28.38	28.34	28.30	28.27	28.23
1	28.19	28.15	28.12	28.08	28.04	28.00	27.97	27.93	27.89	27.86
2	27.82	27.79	27.75	27.71	27.68	27.64	27.60	27.57	27.53	27.49
3	27.45	27.41	27.38	27.34	27.30	27.27	27.23	27.19	27.16	27.12
4	27.08	27.04	27.01	27.97	26.93	26.90	26.86	26.82	26.78	26.74
5	26.71	26.67	26.63	26.60	26.56	26.52	26.49	26.45	26.41	26.38
6	26.34	26.30	26.26	26.22	26.19	26.15	26.11	26.08	26.04	26.00
7	25.97	25.93	25.89	25.85	25.82	25.78	25.74	25.71	25.67	25.63
8	25.59	25.56	25.52	25.48	25.44	25.41	25.37	25.33	25.30	25.26
9	25.22	25.18	25.15	25.11	25.07	25.04	25.00	24.96	24.92	24.89
200	24.85	24.81	24.78	24.74	24.70	24.67	24.63	24.59	24.55	24.52

Formula:

$$\text{percentage of voids} = \frac{266 - \text{wt. of 100 c.c. of sand}}{2.66}$$

NOTES

Temporary Hinges on the reinforced-concrete Warrington Bridge, at Warrington, England, are shown in the accompanying view, from "Concrete and Constructional Engineering," Dec. 1913. This bridge, which is now about completed, has a clear span of 134 ft. and a width of 80 ft. The arch system is made up of ten reinforced-concrete ribs, each 4 ft. 6 in. deep and 3 ft. 9 in. wide at the springing line, and 2 ft. 6 in. deep and 3 ft. 9 in. wide at the crown. These ribs carry spandrel columns and walls which support a reinforced-concrete slab on which the wood-paved roadway is carried. The ribs are reinforced according to the Considère system, which uses spiral hooping with longitudinal rods in the upper and lower plane of the arch ribs. In its completed state each rib is carried down to a solid bearing on the abutment, but in construction the skewbacks are built as shown in the view, that is with the spiral reinforcing left bare for a few feet above the skewback, and the thrust carried through a shallow section of concrete. These hinges are concreted solid after the centers have been struck and the ribs have borne

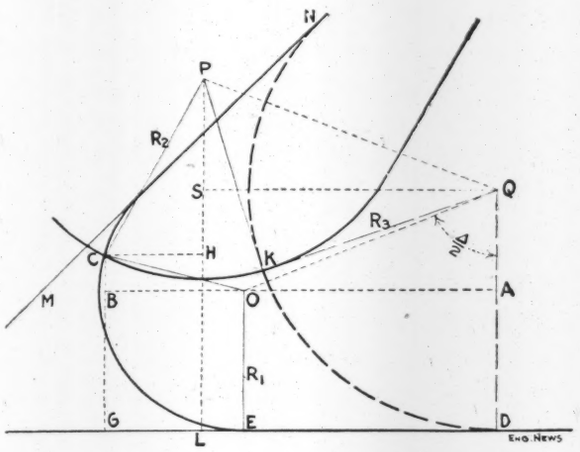


Temporary Hinges in Warrington Bridge, England

...whole load of the structure for several weeks. Similar ... are also provided at the crown.

...ttention is called to the irregularity of the spiraling and ...probable difficulty in concreting around it so that it will ... the way the designer intended.

Curve Intersection Problem—Referring to the problem ... by B. L. G. Rees in our issue of Feb. 26, p. 467, Prof. ... Frank Allen, Boston, recommends working with rectangular ... so far as reasonably possible. In this procedure the ... and progress of the calculation are always apparent. The fact that more than one function of a particular angle is employed does not introduce complication, since log sines, ... and tangents are found on the same page of the tables. Prof. Allen outlines the application of the method to the ... in hand as follows:



Given: A curve of known radius OE = R_1, with center at O, connecting the known tangents MN and GD, with intersection angle Δ; a second curve of known radius PC = R_2, with center at P, cutting the first curve at C, at an angular distance EOC = β along the first curve from its point of tangent; the two curves make the known angle PCO = ϕ with each other at their intersection C. A new curve of given radius KQ = R_3 will replace the first curve and connect the same tangents MN and GD; its center will be at Q.

Required: Angles CPK and DQK to fix the point of intersection K of the third curve with the second curve by giving the angular distances along both curves.

Solution:

OQA = ½ Δ, AQ = $R_3 - R_1$; Solve for OA.

COB = $\beta - 90°$, OE = R_1; Solve for OB and CB.

CPH = $\beta - \phi$, CP = R_2; Solve for PH and CH.

Combine OE, CB, PH; also OA, OB, CH, and thus find coördinates PL and LD = SQ, also PS. From PS and SQ find angles SPQ and PQS; also side PQ. Next, having given three sides R_3, R_3, PQ, solve oblique triangle PQK for angles PQK and QPK. Then

$90° + PQS - PQK = DQK$

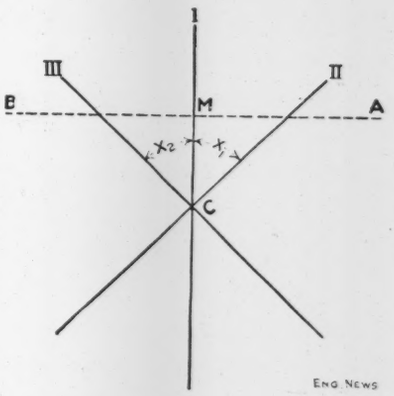
$CPH + SPQ - QPK = CPK$

these two being the required angles.

Accurate Method of Determining Latitude with an Ordinary Engineers' Transit—The method commonly used to make an accurate determination of latitude is by observing the difference in altitude of two stars crossing the meridian at nearly the same distance either side of the zenith within a short interval of time. This method, known as the Talcott-Herrebow method, is described in nearly all the textbooks and is familiar to all who have had to do geodetic surveying. Precise determinations by this method are made with a transit instrument carefully adjusted so that the line of collimation moves in the plane of the meridian and with a filar micrometer at the eyepiece for measuring fine differences in altitude.

When applied to an ordinary engineers' transit where differences of altitude have to be measured by a gradienter screw, the error due to an inaccurate screw may be considerable. To avoid the use of a gradienter screw altogether a Japanese engineer, Kiyofusa Sotome, has devised a simple method which gives results within approximately 5", using an ordinary surveyor's transit. This method has been brought to our attention by Karl G. Habermehl, Geodetic Expert of the Keuffel & Esser Co., 627 Washington St., Hoboken, N. J.

Instead of a micrometer two cross-hairs are added to the



reticule, which cross at the line of collimation and are at approximately 45° to the vertical hair. Referring to the accompanying figure, suppose the instrument set up in the meridian and pointed approximately at one of the stars. The star in passing the field of vision in the direction AB crosses the hairs II and III at recorded times t_2 and t_3. The vertical distance CM from the axis of collimation to the point on the vertical cross-hair where the star crosses is computed from the formula:

$$CM = \frac{15\,(t_3 - t_2)\cos\delta}{\tan x_1 + \tan x_2}$$

Where δ is the declination of the observed star and x_1 and x_2 the angles of the diagonal cross-hairs as shown in the figure.

Assuming t'_3 and t'_2 as the times of crossing for the second star we have

$$L = \frac{\delta + \delta'}{2} + \frac{15}{2}\,\frac{(t_3 - t_2)\cos\delta - (t'_3 - t'_2)\cos\delta'}{\tan x_1 + \tan x_2}$$

+ correction for level by bubble.

+ difference in refraction.

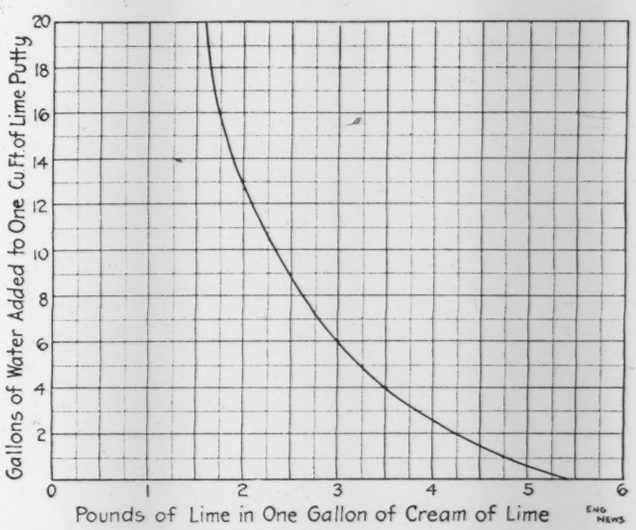
Care must be taken to have the vertical axis as truly vertical as possible as well as the horizontal axis truly horizontal, for a slight inaccuracy in setting means an appreciable error in the results In reversing the telescope about its vertical axis to make the second observation care must be used not to change the altitude setting of the telescope.

The Amount of Lime in a cream of lime mixture is a matter of great importance in stucco or plastering work, but it is somewhat difficult of determination. The accompanying diagram devised by Cloyd M. Chapman, of Westinghouse Church, Kerr & Co. was given in a paper on the subject presented to the recent annual meeting of the American Concrete Institute. The table was prepared from data on eight different high-calcium limes purchased in the New York market. Mr. Chapman states that the figures may need confirmation for limes in other parts of the country.

The curve gives the pounds of lime in each gallon of cream of lime, this amount varying with the gallons of water which have to be added to a cubic foot of the lime putty to make the cream of lime. The curve is given in this form because the workmen are usually provided with a lime putty from which the cream of lime used on the work is prepared. The table enables him to meet the required specification as to the amount of lime to be included in his mortar.

To quote the paper:

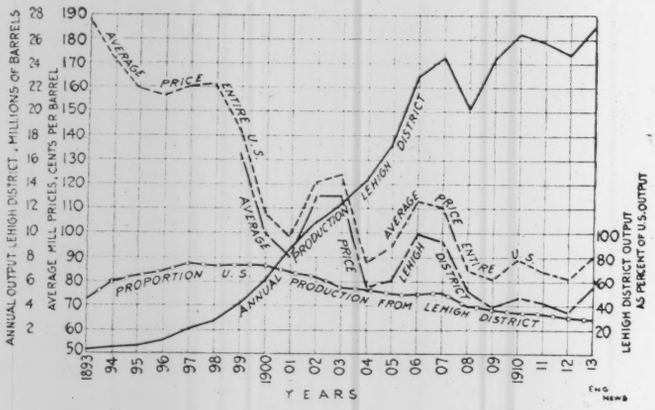
"Suppose, for instance, that the plasterer prepared enough lime putty in advance to do a job of stucco. When ready to use it he reduced 5 cu.ft. of the putty to a cream by adding 50 gal. or 1 bbl. of water or 10 gal. per cu.ft. The resulting cream, from the curve, would have 2.3 lb. of lime per gal.



"Suppose that with this cream the plasterer wished to add 10% of lime to a two-bag batch of cement-sand mortar. This would mean that he would want enough of the cream of lime to give 19 lb. of CaO. As this cream contains 2.3 lb. per gal. he would have to add 8¼ gal. or 2¾ buckets of 12 qt. each."

Production of Portland Cement in the Lehigh District, Penn.—The accompanying curves were sent to us by H. V. Schreiber, managing engineer of Sellers & Rippey, consulting engineers, Stephen Girard Building, Philadelphia, Penn. The curves are a graphic analysis of the output of and price of portland cement from 1893 to 1913, and the corresponding prices. He comments upon the curves as follows:

"The decline in price and the corresponding increase in output from 1899 to 1901 mark the introduction of the rotary kiln. A large part of the demand in 1906-1908 was caused by the reconstruction work in San Francisco following the earthquake and fire.


ANNUAL OUTPUT, LEHIGH DISTRICT, MILLIONS OF BARRELS
AVERAGE MILL PRICES, CENTS PER BARREL
LEHIGH DISTRICT OUTPUT AS PERCENT OF U.S. OUTPUT
AVERAGE PRICE ENTIRE U.S.
AVERAGE PRODUCTION LEHIGH DISTRICT
AVERAGE PRICE LEHIGH DISTRICT
PROPORTION U.S.
ANNUAL PRODUCTION FROM LEHIGH DISTRICT
YEARS
ENG. NEWS

OUTPUT AND PRICE CURVES OF PORTLAND CEMENT

"The decreasing importance of the Lehigh District is explained by the development of other fields, in the South and West. The Hudson District is cutting severely into the New York and New England trade. The decrease in output of the Lehigh District shows that the home market has not been developed to a point where the output of the mills can be absorbed as the outlying territory has been taken over by local plants.

"The steady downward tendency of the average prices, especially in the past few years, indicate that greater economies will have to be practiced or the price boosted. The price in the Lehigh District has been dropping away more and more from the average for the country."

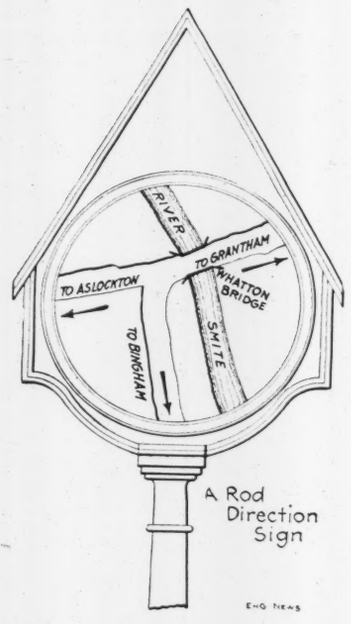
Wood-Block Pavement Failures at joints over tie-rods in street-railway track such as that described by Rodney C. Davis in "Engineering News," Jan. 29, p. 260, and failure along the head of the rail due to expansion, may be avoided, according to E. J. Shutt, Consulting Engineer of the Phillip Carey Co., Cleveland, Ohio, by the use of expansion joints made of "Elastite," a patented material manufactured by the Carey Co. The pavement failure described by Mr. Davis was due to the ship-lapped form of block used to bridge over the ½-in. tie rods. "Elastite" is made of alternate layers of a saturated wool felt, which is coated and bonded together by heavy coatings of an especially compounded asphalt. It is made in strip form from ⅛ to 2 in. thick and in widths up to 12 in., 5 to 18 ft. long. Mr. Shutt states that the wood blocks should be laid with a ¾-in. opening over the tie rods, and this space stuffed with a strip of "Elastite." A similar strip along the rail web is designed to prevent the blocks bearing against the rail head from buckling. "Elastite" is claimed to have better wearing qualities than a simple bituminous filler.

Stopping a Leaky Seam in a Corrugated Steel Pipe—A corrugated steel pipe crossing the wide bed of an intermittent stream as an inverted siphon leaked at one of the joints where by an oversight in manufacture one length of pipe had not been soldered as were the others. The water spurted out of the leaky seam and threatened destruction of the supporting earth bank. The leak had to be stopped quickly as there was no means of stopping the flow of water in the ditch in time to avert the disaster. The manager of the ditch had heard that a leak in a steam boiler could be temporarily checked if a small quantity of horse manure was introduced within the boiler. He reasoned that it would also be effective in the present case. A quantity of that material was close at hand and he determined to try it. Two wagon loads were pitched in at the upper end of the siphon and within a few minutes the leak was entirely checked. The siphon was operated the entire season with no further repair.

The repair of siphons in this way is not in general advocated, but the idea may be worth while on occasions. [C. V. Mann, Boulder, Colo.]

Table of Weights of Angles—As given in the steel-mill handbooks, the tables of weights of angles are spread over several pages. This necessitates, in estimating steelwork—particularly in mill buildings, which involve mainly angles—the constant turning of pages to get from 3x2 angles to 6x4 or to 3½x3½ angles, etc. The tables are easily condensed into a single table of one page by an expedient suggested by Alex. Broadhead, of Kennett Square, Penn. The argument in this table is the sum of the legs, e.g., 6 in. (representing either 3½x3½ or 3x3 angle), 10 in. (representing 5x5 or 6x4 angles), etc. The vertical columns of the table are for the different thicknesses. In using the table, the size of angle must be converted mentally into the single figure of the sum of the legs, but this is automatic and does not make the work slower or harder.

Road-Direction Map Signs to be placed either at road intersections or to protect dangerous crossings, at some distance from the intersection, have been devised by C. H. Cooper, borough engineer of Wimbledon, England, in conjunction with his assistant, Mr. Bowen. In a letter to the London "Surveyor" of Feb. 6, Mr. Cooper stated:



It occurred to me that if the plans of the intersection were substituted for arms, far better information would be afforded approaching traffic. Thus a plan 3 ft. diameter, to a scale of 8 ft. to 1 in. will embrace 288 ft.; on this a 40-ft. road appears 5 in. in width, and allows 4-in. letters, easily readable at a distance of 200 ft. Where a post could be observed from each road, plans can be placed on such post facing these roads, and so avoid a multiplicity of posts. If transparent faces are used with a light, these plans might be illuminated at night.

A proposal that the post bearing this sign might also be used to bear another one giving "distance to the nearest doctor" contains a grim suggestion of possible accident which is not at all surprising in view of the fact that no longer ago than December, 1913, three persons were killed by an automobile running into the wall or railing of the bridge shown on the accompanying road-sign map. The automobile was coming from the direction of Bingham. Mr. Cooper states that such signs as he proposes, guarding the approaches to this bridge, would have given the driver knowledge of "the road to take and the existence of the bridge where the accident took place."

The Camp and Commissary Arrangements for field engineering parties and floating gangs of the Nashville, Chattanooga & St. Louis Ry. are about as follows: Supplies are bought in various quantities as needed, at the nearest point, and the provisions are housed usually in a small tent next to the cooking tent, which latter serves also as a mess tent. Besides these, there is an office tent, and sufficient tents to house the party. The men are supposed to walk to and from work, and hire teams to move camp. Except on rare occasions no teams are retained with the party. The floating gangs of the company's labor forces are housed as a rule in caboose cars, each gang having eight cars, as follows: Foreman's car, cook car, dining car, three sleeping cars and two tool cars. Each sleeper has 6 to 8 double-deck bunks, and accommodates 12 to 16 men, but there are rarely more than 12 men to the car. The men are fed by the railway company, supplies being bought in Nashville and distributed to the several gangs as required. The company also furnishes a cook and helper for each gang, and the cost of board ranges from 28 to 35c. per head per day.

Bridge gangs receive higher pay and feed themselves. Section men are provided with houses for themselves and their families, and they also feed themselves.

Editorials

We are informed that the stock-selling engineering concern which was noted in our issue of Feb. 19 and again in our last week's issue, as having been raided by the public authorities, has now transferred its operations to Chicago. The local post-office officials declined to interfere until complaint should be made by someone who had lost money, and the newspapers could not well stop the "want" advertisements as they were sent in from substations. The facts of the case and the character of the business have been brought to the attention of the agents of the building in which offices had been established. We can only repeat our former caution that engineers and others seeking employment should be suspicious of offers from any concern requiring the purchase of stock.

Traffic through the Suez Canal

Traffic through the Suez Canal, which has shown an almost continuous growth ever since its opening, was less in 1913 than in 1912. Had this falling off occurred a year later it would have been ascribed, of course, to the opening to traffic of the Panama Canal. As it is, the cause must be sought elsewhere, and it is doubtless found in some slackening in the phenomenal activity in international trade which prevailed in 1912.

What makes the falling off in traffic quite remarkable is that it occurred just after the rate of toll on vessels passing through the canal had been reduced, which might have been expected to stimulate the traffic. The figures for the number of vessels, total tonnage and total receipts from tolls for the last three years are as follows:

	1911	1912	1913
Number of vessels	4,969	5,373	5,085
Total tonnage	.....	28,008,945	27,737,180
Receipts, francs	131,024,000	132,912,000	122,980,000

The Institution of Civil Engineers Opposes Engineering Competitions

In England, as well as in the United States, frequent instances occur of municipal authorities which have engineering work to do inviting competitive bids from engineers for performing the work. It appears that the practice has become common enough in England so that the Institution of Civil Engineers has seen fit to take notice of it by issuing the following circular to its members:

Dear Sir:

The Council have had their attention drawn for some time past to a practice which exists among local authorities, of inviting by advertisement engineers to submit in competition with others their terms for preparing plans and proposals for certain engineering schemes, accompanied then or perhaps at a later stage by estimates of the cost of the works proposed.

In the opinion of the Council such a proceeding is very undesirable in the best interests of the public authorities themselves and is derogatory to the enginering profession. The Council desire to express emphatically the repugnance with which they regard the practice in question. They have every confidence that the members of the Institution will support them by declining to respond in any way to such advertisements as those alluded to.

The Council have informed the Local Government Board of their action in making this communication to the Institution.

About the only criticism that can be made of this circular is that it might well have been supplemented by an offer of aid to municipal authorities desiring to employ an engineer by recommending to them men who would be especially competent for the particular work to be undertaken. It must be remembered that the average alderman or member of a town council has very little acquaintance as a rule with engineering matters and with engineers. One reason why he is ready to award engineering work to the lowest bidder is because he realizes his incompetence to judge of the ability of one engineer over another for the work in hand.

The Small Concrete Mixer

It is not so many years since certain conservative engineers regarded the mechanical mixing of concrete as a dangerous innovation, possibly applicable to certain kinds of unimportant concrete work on account of the saving in cost, but not to be thought of in the major operations where strength of concrete was a prime necessity. Even so recently as 1907, a standard work on concrete with rare progressiveness admitted that "the best forms of mixers now on the market give results quite equal to the best hand work." Today, the superiority of the mixer admits of no discussion; engineers have rather tardily recognized that muscle cannot compete with machinery.

While the concrete mixer has become an indispensable adjunct to construction, there are any number of pieces of work where the amount of concrete to be used at any one place is so small as to make a mixer inadvisable both from an economic and operative standpoint, and in these cases the old hand mixing is still resorted to, although it is highly probable that the concrete so mixed is not as good as that mixed by machine. For such instances there has recently come onto the market a great number of small mixers of admirable design and advantageous price At the recent cement show at Chicago, about the only feature worthy, on account of its novelty and interest, of the attention of the engineer and contractor already fairly conversant with concrete work, was the great number of these small mixers. At least a dozen different makes were on exhibition.

Certain features are common to all of these mixers, although naturally there are points of individual superiority. Taken as a class, however, this small mixer is of the batch type, capable of mixing from ¼ to ⅓ cu.yd., and is mounted on an easily portable truck together with a gasoline engine drive and, occasionally, a loader and unloader. Their price varies from $150 to $400, complete. When this type was first put on the market, it suffered from a marked fragility, so great, in fact, that it earned the term "tin-pan mixer," but the latest exhibit in Chicago showed that the manufacturers are beginning

to realize the necessity for staunch construction. Some of the machines, in fact, seemed to be of even more solid design than is common in their larger predecessors.

While the obvious field of the small mixer is the concrete sidewalk, house, block factory, etc., where isolated concreting is obligatory and where the large mixer would be too expensive, it would seem to have a very real place in the larger kinds of concrete work. On every big contracting job concrete has often to be laid in locations and at times when it is extremely inconvenient to use the main mixing plant and recourse must then be had to hand mixing. It is quite within belief that for just such occasions one of these small self-contained and portable concrete mixers would soon pay for itself, not only in the character of the concrete produced, but in the actual cost of making and placing. It has its obvious limitations, but it is a very interesting development in the concrete field and, judging from the great number of manufacturers who have taken it up, it promises to be a live one.

The Honors Bestowed on Col. Goethals

On the evening of Mar. 3, at one of the largest public dinners ever held in Washington, President Wilson presented to Col. George W. Goethals the gold medal of the National Geographic Society in recognition of his work on the construction of the Panama Canal.

In New York City, on the following evening, Carnegie Hall was crowded with a distinguished company to witness the presentation of a special medal of honor prepared by the Civic Forum and presented to Col. Goethals on behalf of that organization by President Finley, of the University of the State of New York.

At the Hotel Astor, on the following evening, a thousand members of the Economic Club attended a dinner in honor of Col. Goethals and witnessed the award to him of the medal of the National Institute of Social Science.

Every member of the engineering profession may well be gratified when such notable honors are conferred upon a great engineer in recognition of the work which he has done. This is especially the case because it has frequently happened that the public honor for the successful conduct of a great work has been given to the promoter or the financier in charge instead of the engineer.

It is the case more often than not, too, at the present day, that engineering works of great importance and difficulty proceed from beginning to end almost unnoticed by the general public, which has indeed become so accustomed to marvellous feats of engineering that it is apt to accept them as a matter of course.

It is an old proverb that "republics are ungrateful," but a modern addendum to it would be "corporations know no gratitude." It is easy to see that the public interest in and appreciation of engineering works is far greater if the work is under public control than if it is done by a corporation. It will hardly be questioned that the public interest in the Panama Canal work and appreciation of the engineers who have carried it to success is tenfold greater than would have been the case had the canal been built by a private corporation.

During the decade since the Panama Canal work began, American railway companies have completed a dozen huge enterprises, such as the Pennsylvania Railroad terminals in New York and Washington, the New York Central terminal in New York City, the Chicago & Northwestern terminal station at Chicago. Of how few of these great works has any adequate public celebration been held? Of how many of them has the public ever heard any word concerning their engineering creators? We doubt whether these engineers have been honored by so much as a line in the annual report to the companies' stockholders.

We may well look back with some regret to the custom of an earlier day, when the driving of the last spike on a railway line, or the completion of an important bridge or station, was made the occasion for a fitting celebration, at which it very frequently happened that the engineer responsible for the work came in for some share at least of public honor. It is, of course, true that the yardstick by which we measure performance has grown apace with advancing years. A million-dollar job, which was a nine days' wonder generations ago, is today a matter of ordinary routine in connection with the operation of a great railway or industrial corporation.

There is reason to believe, however, that even in work done by great corporations there would be advantage in returning to the custom of an earlier day. If the corporation managers take the great achievements of its staff as a matter of course, is there any wonder that the public does the same? Suppose a great railway or industrial corporation were to inaugurate the custom of holding public celebrations of the completion of any important piece of work and paying due honors to the men who have carried them to success. We believe such occasions would foster a better appreciation of the work which the much reviled corporations are doing for public benefit. Besides this, such generous recognition of the engineers who have amply earned honors would add greatly to the *esprit de corps* of the whole working force.

The honors paid to Colonel Goethals are of great value to the engineering profession from another point of view. The bestowal of these honors in recognition of his great work at Panama serves to emphasize to the public the high value of the trained and experienced engineer who combines with his technical knowledge a high order of executive ability.

We say this is an object lesson of great value because there is a widespread fallacy abroad that any bright man is competent enough to perform successfully any job in which a popular election or a political appointment may land him. Those who believe this fallacy, believe also, of course, the other fallacy that a high salary to any man is wrong since there are plenty of other men equally competent who would have been glad to have undertaken the work. Of course those who hold to this belief, hold also that one engineer is as good as another of equal training and experience. The element of personality which enables one man to win distinguished success in a task where another wholly fails is overlooked.

Those who have shared responsibility for important work involving the efficient management of large forces, however, know by experience how rare are the men of demonstrated ability to successfully perform the big tasks of the present day. The head of a great financial institution in New York City recently declared that he did not know where to find men of sufficient competence to fill positions of large responsibility in the concerns in which his institution was interested.

mere argument, however, will teach the public the fallacy of the idea that any man is good enough for any job. It is only such object lessons as the success of Colonel Goethals at Panama that teach the public the lesson that great men are needed to conduct great works.

It is of especial interest at the present time, too, to recall the lurid stories concerning the difficulties of the work at Panama to which the newspapers gave such wide circulation all through the early years of the canal work. Disconcerting though these stories were at the time, they have so magnified the difficulties encountered at Panama that even those who hold to the theory that any man is big enough for any job do not venture to suggest that any other engineer or executive head could have done as well at Panama as Colonel Goethals.

The Constitutional Amendments in the American Society of Civil Engineers

The great men who framed the Constitution of the United States, with deliberate intention made the process of its amendment so complicated and difficult that during the century and a quarter since the adoption of the Constitution, only a few of the many changes which have been agitated concerning that document have ever been actually carried into effect.

Those who framed the present constitution of the American Society of Civil Engineers apparently held a similar idea—that it was wise to make changes in the instrument as difficult as possible. Hence they required that not merely a majority vote but a two-thirds vote should be required to make valid an amendment to the Constitution. If, therefore, any proposed change in the constitution meets with any considerable opposition among the membership, its chances of adoption are small. With a clear understanding of this, many of those who favored the amendments to the Am. Soc. C. E. constitution on which a letter ballot was counted at the meeting last Wednesday night, felt from the beginning that the success of the amendments was doubtful, in view of the strenuous opposition of many members.

The result of the ballot, which is given in detail elsewhere in this issue, shows that the amendments would have been defeated even had the constitution permitted a change by merely majority vote. On the first two proposed amendments, however, the votes for and against were not greatly different in number.

It would be interesting if a concrete expression were made as to just what the vote signifies. That is to say, as to just what was in the minds of the members who voted for and voted against these proposed amendments. In the first place, since the number who actually voted was only about half the Society's total voting membership, it is evident that fully half the members did not take the trouble to vote. Probably most of those who failed to return the letter ballot did so either because they did not understand the question at issue and preferred not to commit themselves upon it or else because they have comparatively little interest in the conduct of the society any way.

So far as the amendments themselves are concerned, they are, of course, merely means to an end. The viewpoint of those who voted for the amendments is in general, we take it, that the Society should broaden its functions and undertake various tasks which it has not hitherto attempted. The viewpoint of those opposed the amendments, on the other hand, is, we presume, that they approve the conduct of the Society in the past and oppose any action that might be construed as criticism of that conduct. This idea in fact was expressed by Past-President John F. Wallace, in making a motion to table the amendments at the recent annual meeting.

The divided vote on the constitutional amendments, should not by any means be construed as indicating that the Society members who take an active interest in its affairs are split into two irreconcilable opposing camps. Nine-tenths of those who cast the ballots which were counted last Wednesday would agree, doubtless, on the proposition that everything possible should be done to make the Society of the greatest usefulness to the profession. It is quite possible that had the effect of the proposed amendments been clearly understood by the membership, apart from all questions of personality, they would have met with substantially unanimous approval.

It is interesting, by the way, to contrast the different methods by which the matter of official nominations for officers in the Society has been handled in the American Society of Civil Engineers and in the American Society of Mechanical Engineers. In the Civil Engineers, there is under the present constitution most elaborate machinery provided for the selection of a nominating committee, chosen by geographical districts covering the entire country, so that by no possibility can the nominating committee be bunched in a given locality and unduly favor the selection of officers to represent that locality alone. Further than this, the constitution itself prescribes that the Society's directors shall be geographically distributed over the country. We know of no other technical society in which any such elaborate provisions have been made to insure that the officers elected shall be truly representative of the society at large.

In the American Society of Mechanical Engineers, on the other hand, the official nominating committee is appointed by the president of the society. The only provision respecting it is that none of its members may be chosen from those then holding membership in the Society's Council. By custom, however, a Past-President of the Society is made the chairman of this committee. The nominating committee so appointed has full authority to make its own selection of nominees for Society officers. A safeguard which has proved amply sufficient to prevent any attempt to keep control of the Society in the hands of an "inner ring," is that any twenty members of the Society may organize as a special nominating committee, select a list of candidates for the vacant offices, and such nominations must be sent to the members along with the regular ticket nominated by the officially appointed nominating committee.

There is, indeed, a similar safeguard embodied in the constitution of the American Society of Civil Engineers, under which any twenty-five members of the Society who choose to take the trouble, can nominate a full ticket for the Society's officers and have it sent out to ballot along with the regular nominations.

Bearing this in mind, the defeat of the constitutional amendments, recently voted upon, loses a large part of its importance. There is certainly machinery enough in the present constitution to carry out the will of the majority of the membership, whenever it chooses to put that machinery in operation.

Letters to the Editor

Publicity for the Engineer

Sir—After reading the articles under the above head in recent numbers of ENGINEERING NEWS (*ante* pp. 176, 201 and 474), the writer sorrowfully realizes what a conservative profession engineering really is.

Away back in 1884, just 30 years ago, the writer on his return from Mexico, where he was associated with the lamented A. W. Wellington on the Mexican Central Ry., read a paper on the "Railway System of Mexico," before the Engineer's Society of Western Pennsylvania.

Realizing that Mexico was a *terra incognita* and wishing to excite interest in the subject of the paper, the writer prepared abstracts of it, couched in popular style, which were furnished to the daily papers and published the day after the meeting. Before this meeting there were very few people in Pittsburgh who had ever heard of the Engineering Society, and fewer that there were any railroads in Mexico. Unfortunately, other authors of papers failed to use similar methods of publicity and the society soon fell into that condition of "innocuous desuetude" mentioned by your correspondent, Mr. Poe. Happily that condition does not now exist.

At that time, also, the Nicaragua Canal was prominent in the public eye and I furnished a leading daily newspaper of Pittsburgh articles readily understood and derived from official sources. I may add that these articles were eagerly sought after and always printed in full. As Mr. Poe forcibly says, in all of the large cities there are engineering enterprises in progress which would make good copy for Sunday magazine sections. Of course, the articles must be written in a style readily understood by the general reading public and free from the technical language usually employed in engineering papers. If engineers crave for publicity and desire that the world should know of their achievements, it develops upon them to take the initiative and not wait for reporters to do it for them in an inefficient manner.

EMILE LOW.

606 Ashland Ave., Buffalo, N. Y., Mar. 2, 1914.

Sir—The communication under the above caption in ENGINEERING NEWS, of Feb. 26, 1914, brings up again a subject of never failing interest, because it treats of the development of the sources of personal advancement. That "it pays to advertise" has been abundantly proved; and this being so, why do not all engineers advertise? Asking that question and insisting upon frank answers, will bring to light the real reason why more engineers do not follow the advice of Mr. Poe in bringing their names before the public.

One of the principal reasons why engineers, particularly the younger ones, do not write for the technical journals, is that they are not allowed credit for their own work, and do not care to write for the exclusive benefit of the head of their department. It is becoming more and more a fact that the chief engineer in charge of important works is less an engineer than a financial and executive head; often he is not an engineer at all. The less these men know about the actual technical work under their control, the more jealous are they of their reputations, insisting upon receiving all credit, and excluding anyone else. Thus, if a young engineer designs a bridge which forms a part of a large engineering work under a well known engineer, he will not be allowed to describe it in print with any credit to himself. This is not a theory, but a condition which confronts most engineers working on large undertakings under illustrious heads. The most available form of advertising is not open to them.

From another point of view, one gets the idea that really good technical literature is the poorest form of advertising. My personal experience has been that the man who is capable of producing serious and useful articles for technical journals does not profit by them. Whether this is due to the fact that he is temperamentally unfitted to take advantage of advertising, or because serious articles are not good advertising matter, I shall leave to individual judgment.

To write good technical papers, one must have something to say, or he will fall into the class of verbose platitudinizers known as "windjammers," unworthy of the attention of the technical world.

Quite different, however, is the advertising appeal to the unscientific mind through the columns of the daily press. He who can have his engineering miracles held up to the awful admiration of the masses will undoubtedly progress, but along paths which his more serious brother would refuse to traverse. To advertise one's profession, like a safety razor or a hair restorer, may bring fame and money, but the real technical man cannot do it, and the one who does do it is not a real technical man, but a "business man." Then, which shall it be; engineer or advertiser? I believe one person cannot be both.

HENRY A. MARVIN.

Yonkers, N. Y., Feb. 28, 1914.

Causes of Derailments of Locomotive Tenders

Sir—In your editorial on a derailment accident, in ENGINEERING NEWS, of Feb. 12, I think your suggested remedy of lowering the center of gravity of tenders is a correct one, but does not go far enough to insure safe running at high speed.

I have had some experience in investigating such accidents, on a straight track, and the conditions and results have been so similar that I feel safe in reaching conclusions.

Twelve years ago when I was locating engineer of the Missouri Pacific Ry., I was on a M. P. train, west of Little Rock, Ark. Running at 40 miles per hour, the rear truck of the tender was derailed, finally bunching the ties and stopping in 1000 ft. I examined all the con-

dition carefully. There was no failure of any of the equipment, everything belonging to trucks being in place. The track was in fair condition and the point of derailment easily located.

About four years ago, when on a train of the Pere Marquette R.R., from Detroit to Toledo, the same sort of derailment of tender rear truck occurred at 50 to 60 miles per hour. The train ran about 1200 ft., stopping with the rear truck against the forward truck of the baggage car. All parts of the derailed truck were in position, and the track at the point of derailment was in good line and surface.

I have concluded that the "surging" of the trucks was the direct cause in both of these cases and in others I have observed.

In using the three-wheel velocipede, trackmen know it is necessary to set the front wheel at a certain slight angle with the gage line, the wheel lining toward the center line of the track usually, and no matter how carefully this is done, at a certain speed (there being practically *no* wheelbase) the car surges, and may easily derail on straight track; being safer on a well lined and elevated curve. I have observed the same thing in hand cars; at about 15 miles per hour with ordinary loading (4 or 5 men), they run very steadily. On down grade, at 20 miles per hour, or more, the same surging occurs as in the case of the velocipede.

The short wheelbase of tender trucks is the direct cause of such derailments. Surging, helped by the unsymmetrical water loading, precipitates them. For all trains at speeds exceeding 30 miles per hour, I think the only remedy is in a longer tender by at least 6 ft., allowing an increase in each truck-wheel base of 2 to 3 ft. This would make the lowering of the center of gravity more feasible. These changes, with the addition of the swash plates you suggest, would very materially reduce the number of derailments.

A. J. Wenzell,
Chief Engr., Michigan & Chicago Ry.
Kalamazoo, Mich., Feb. 25, 1914.

The Design of Railway-Bridge Floors

Sir—In your comment on the Interstate Commerce Commission report of the derailment of one of this company's trains near Buckatunna, Miss., in your issue of Feb. 12, you make the following statement:

"It was not stated whether there was any blocking between the ties to prevent their being bunched by derailed wheels, and it is probable that this very important feature of the trestle was omitted.

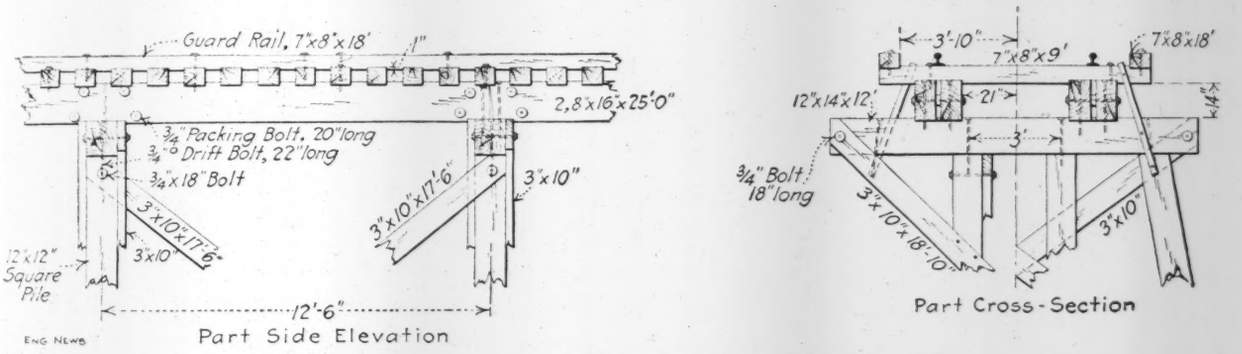
The outside guard timbers were fastened by lagscrews to every second tie.

For your information I inclose a copy of the Mobile & Ohio R.R. Co.'s standard plan for pile trestles which represents clearly the type of floor that was on the trestle in question. You will see that the guard timber is dapped one inch over each tie, thus conforming to the common standard on most American railways.

However, this presents an interesting problem. What would have happened had there been no wood blocks to prevent the bunching but in their stead each tie had been fastened on each end to the stringers and to the guard timber with a lagscrew, making four lagscrews per tie?

H. Austill, Jr.,
Bridge Engineer, M. & O. R.R.
Mobile, Ala., Feb. 17, 1914.

[In our opinion the blocking between the ties made by dapping the guard timbers over them is a better safeguard against the bunching of the ties by derailed wheels than fastening the ties to the timbers by drift bolts or lagscrews, and we believe it is generally so regarded by bridge engineers. We reproduce a portion of the standard plan referred to above showing the trestle floor.—Ed.]



Standard Floor for Pile Trestles, Mobile & Ohio R.R.

New Photo-Reproduction Systems recently introduced provide for the reproduction of tracings on tracing cloth and the photo-reduction of blueprints. The reproduced tracings are of the same size as the originals and blueprints can be made from them in the usual way. As no solution is used there is no damage to the original or the copy. This process is considered specially desirable at the present time, in view of the fact that the railway valuation board of the Interstate Commerce Commission is requesting the railways to furnish tracing-cloth copies of all their original tracings. It will give exact reproductions without the time, labor and checking involved in making new tracings. The initial copy made by this process is expensive, but after the plate is made the cost decreases with the increase in number of copies. There is in England the "Ordoverax" process for this purpose, and in France the "Dorel" process. An attempt was made some years ago to introduce the latter in this country, but was unsuccessful owing, it is said, to climatic conditions, since a low temperature and a certain degree of humidity must be maintained.

The photographic copying of blueprints is often desirable, and has been found of special advantage in cases where original tracings have been lost or damaged and only blueprints were available. The original may be of any size, but the size of the copy is limited to 11x14 in., so that very large originals would be copied in sections. Smaller originals can be enlarged. No negative is used, thus eliminating the delay of fixing and washing before printing. The cost is low, and the process takes but a very short time. The copy is made upon a sensitized parchment which can be folded without cracking and will stand rough usage, and from which blueprint or whiteprint copies can be made. Both of these processes are specialties of the American Blue Print Paper Co., of Chicago.

The Recent Storm Damage in Los Angeles

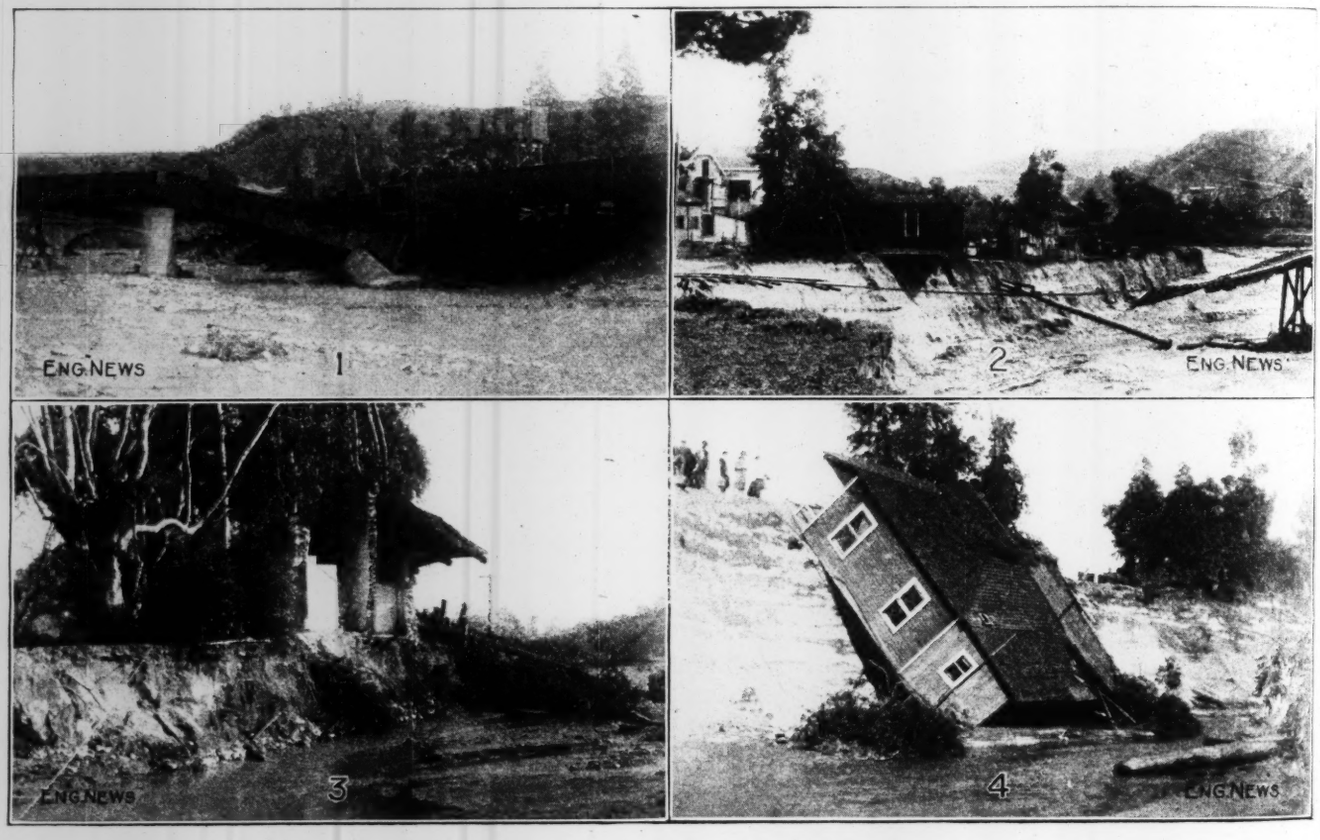
An idea of the great amount of damage done on Feb. 17, 18 and 19, by the floods in southern California, is given by the accompanying views taken in Los Angeles by a correspondent who contributes also the following information.

As briefly noted in ENGINEERING NEWS, of Feb. 21, some 7 in. of rain fell, which, however, of itself does not indicate the cause of the great damage. A severe storm in January had brought the seasonal rainfall up to 15.04 in., or about normal, and this had saturated the ground so that the runoff from the second storm was a much greater percentage than usual.

The worst of the storm was over on the second day, but enough rain fell on the 19th so that the streams remained swollen and continued to undermine the dirt banks of the rivers and arroyos, which had never been protected because of the infrequence of such floods. The river beds are of gravel and the slope great enough to give the water great velocity. The banks are of sand, loam and clay, and they melted like snow under the impact of the swift torrents that raced down the streambeds. It was the velocity rather than the amount of water which was responsible for the great damage.

In Los Angeles, the worst damage was along the Arroyo Seco, where several bridges were wrecked and numerous bungalows carried away altogether. Whole streets along each side of the channel disappeared. Many landmarks have been obliterated entirely. The celebrated Los Angeles pigeon farm, "the largest in the world," located at a bend in the Los Angeles River opposite Elysian Park disappeared as if by magic, only a single row of coops remaining. Several thousand feet of railway sidings along the Los Angeles River were carried away with the high bank on which they rested. All the old car trucks that could be secured were dumped over the bank, where the attack of the water was most severe, in an attempt to stop the destruction. Long stretches of high-tension power lines were thrown down by undermining of tower foundations. All train service was abandoned and Los Angeles was practically isolated. Some 500 passengers from stranded Eastern trains were brought in on two flat cars and two box cars by foot transfer across the San Gabriel River. Fortunately, the city car lines and the lighting and telephone service continued with little interruption. The natural gas lines were cut off.

Many ranches south of the city were flooded; one worth $400,000 and another worth $80,000 had all their improvements ruined. A few lives were lost. Los Angeles Harbor has been seriously damaged by the deposit of several thousand tons of river spoil. In the six counties most affected, some 35 bridges or more were lost. The damage has been variously estimated up to $3,000,000.



VIEWS OF THE LOS ANGELES FLOOD DAMAGE

(1) Bridge of Atchison, Topeka & Santa Fé R.R. Over Arroyo Seco; Concrete Pier Undermined and Tipped Over. Spans at Right Have Been Raised on Timbers.

(2) Remains of Trestle Across Arroyo Seco. Typical of Changes in Real Estate.

(3) Ruins of Comfort Station at Sycamore Grove (Los Angeles City Park).

(4) Bungalow that Was On the Bank of the Arroyo Seco, Just Above the Junction with the Los Angeles River. Showing the Great Amount of Soil Carried Away.

Collapse of a Portion of the Antelope Valley Steel Siphon, Los Angeles Aqueduct

By William Mulholland*

SYNOPSIS—An 8-in. rainfall in three days in desert country where the annual rainfall rarely exceeds 7 in., caused a washout where a 10-ft. steel siphon of the Los Angeles Aqueduct crosses a small creek. The concrete piers supporting the pipe were overturned, the pipe broke at several joints, and the outrushing water emptied the pipe so rapidly that a portion of it collapsed under external air pressure.

The Antelope Valley Siphon has a total length of 21,766 ft., with a maximum head in the center of the valley of two hundred feet. Of the length, 2736 ft., at the north end is of reinforced concrete and also 3314 ft. at the south end. The remaining length is of steel, 10 ft. in diameter. The middle half of the steel is composed of 3/8-in. plate and the two ends are of 1/4-in. plate. All the steel pipe is laid half its depth in the ground.

In the center of the valley, when the pipe was laid, there was a small ravine about 8 ft. deep by 15 ft. wide, which afforded drainage for the entire west end of the valley, and from its appearance indicated that the maximum flow that ever passed there was never in excess of 40 or 50 cu.ft. per sec., the whole country being in an arid desert region where the annual rainfall rarely exceeds 7 in. The pipe was carried across this ravine on two concrete piers, one on each side.

On Feb. 18, 19 and 20, the whole of southern California was visited by a storm that in point of destructiveness was never equaled in this country. As a result the city of Los Angeles was totally isolated from traffic with the outside world for over a week.

The rainfall at a point along the Aqueduct ten miles from the Antelope Valley siphon, was over 8 in. in that period, resulting in a freshet that cut out the small ra-

*Chief Engineer, Los Angeles Aqueduct, Los Angeles, Calif.

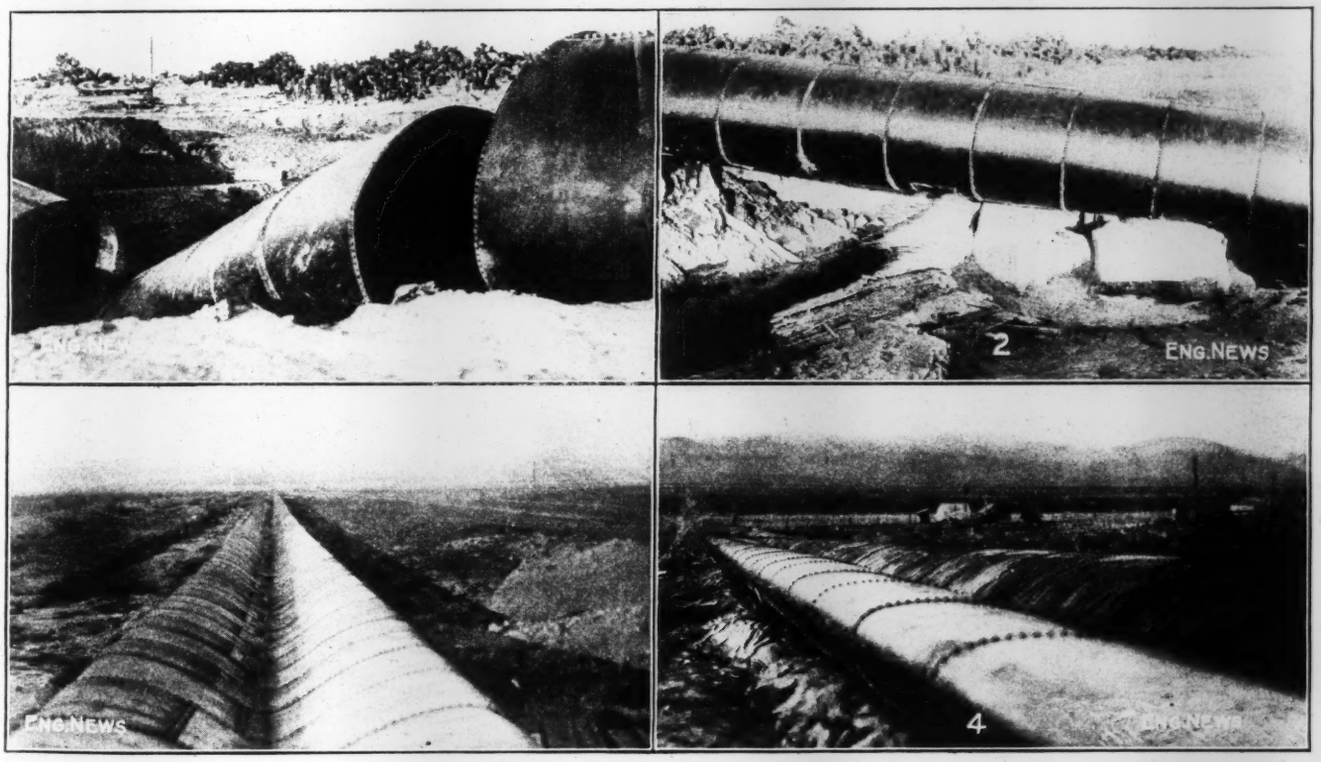


Fig. 1. Break in Steel Pipe at Joints
Fig. 2. Two Overturned Piers and Pipe Displaced
Fig. 3. Looking Down the Line
Fig. 4. Looking Alongside the Line

Views of Failure of Antelope Valley Siphon, Los Angeles, Calif., Taken Feb. 25, 1914

[From photographs supplied by F. C. Finkle, consulting engineer, Los Angeles, Calif. Mr. Finkle, writing five days earlier than Mr. Mulholland sent his account of the failure, expressed the opinion that "the break was due to failure of defective concrete piers under the pipe crossing Antelope Valley Creek, in which [creek] only a small amount of water has flowed the present year, as evidenced by water marks on the banks." Mr. Finkle also expressed the opinion that it would be impossible to restore the collapsed pipe to its original shape by restoring the internal pressure and added that "engineers and manufacturers are of the opinion that the sheets must be cut out and rerolled or new material must be purchased."

Fig. 2 was taken at the crossing of Antelope Valley Creek. The cradles in which the pipe rested face each other in the overturned piers, and the cradle marks may be seen on the under side of the pipe.]

vine above described to a width of 150 ft. and a depth of over 20 ft. This washout undermined the two piers, which still lie on their sides intact in the bottom of the ravine, letting the pipe drop down and tearing three of the round seams apart [see Figs. 1 and 2]. The pipe being in commission delivering water at the time the water rushed out, naturally, with such high velocity that the quarter-inch pipe collapsed to a U-shape. [See Figs. 3 and 4.] The heavier plate remained intact and the reinforced-concrete pipe was also uninjured.

As there is no place where the pipe is bent to a smaller radius than 6 in., it can reasonably be expected that it will be restored to roundness when repairs are made to the broken part in the center of the valley and the pressure turned on. Some of the rivets, without doubt, where the bending is sharp, will have to be replaced. The repairs in the central section are about completed [March 9] and it is expected that the water will be turned through to round out the quarter-inch pipe before the end of the week.

The accident is similar in character to that which happened to the Bull Run pipe, at Portland, Ore.,* some years ago, but will be more easily repaired as that pipe was of the lock-bar type and sustained some damage by rupture of the plates along the bar.

With the exception of this accident and the trivial amount of sand running into the open portion of the Los Angeles Aqueduct near its head in the Owens Valley, no further damage was incurred on the whole 217 miles of the Aqueduct during the most violent storm in this part of the country within the memory of the writer, who has lived here for 37 years. The money loss may run as high as $2500 and the Aqueduct cost $24,500,000.

The Ice-Melting Power of the sun's rays have been measured by Prof. J. Maurer, of the Central Swiss Meteorological Institute as about ¼ in. per hour. He compared the diminution in thickness of ice plates in the sun and in shade.

*See "Eng. News," July 27, 1911, pp. 112-115, for illustrated description of Bull Run pipe collapse, which occurred while the conduit was being tested.—Ed.

Severe Gas Explosions under 23d St. at Fifth Ave., New York City, on March 8

Three heavy gas explosions on the morning of Mar. 8 ripped two large holes in 23d St. at the intersection of Broadway and Fifth Ave., New York City (see Fig. 1). Breaks in the water mains flooded the subway excavation on Broadway to a depth of about 10 ft. The safety vaults of the Garfield National Bank, at 200 Fifth Ave., were flooded, and many plate-glass windows in the vicinity were broken. Eight persons were injured, but none seriously.

The larger hole, on the south side of 23d St., was about 25 ft. long, and extended from the surface car tracks nearly to the curb, as shown in Figs. 2 and 3. The other hole was a little to the west, on the north side of the tracks, and was about 10 ft. long by 7 ft. wide. It reached down to a sewer which was broken by the explosion. The general extent of the street damage is shown in Fig. 1.

The sequence of events appears to be as follows: The breaking of a 48-in. water main under the intersection of Fifth Ave. and Broadway, at 23d St., at about 8 a.m., undermined the gas mains so that they sagged and leaked. At about 11 a.m., a 6-in. gas main exploded, probably from an electric spark. There was a quantity of underground electric wiring at this point. A west bound crosstown car was directly over the point of the explosion,

FIG. 1. GENERAL VIEW OF SCENE OF EXPLOSION, SHOWING PLANKING COVERING SUBWAY EXCAVATION

and the passengers were badly shaken up, although only eight were injured to any extent. It seems probable that the underground contact system of the car caused the spark which ignited the escaping gas.

The second and third explosions occurred between 11 a.m. and 12:30, noon. The flooded areas were pumped out by fire engines. The fortunate fact that the accident occurred on Sunday accounts for the small number of injuries. On week days, this is one of the busiest corners in New York City.

Official investigations were started on Mar. 9.

The Boston Engineers' Dinner

The fifth annual Boston Engineers' dinner was held on the evening of Wednesday, Mar. 4, at the Boston City Club. This annual dinner has become a notable affair in connection with the movement to foster coöperation between the local engineering organization and the local branches of the national engineering societies. The dinner held on Wednesday evening last was conducted jointly by the Boston Society of Civil Engineers, the American Institute of Electrical Engineers, and the American Society of Mechanical Engineers. The committee in charge of the arrangements for the dinner was made up of representatives from each of these three organizations, with N. J. Neall, as chairman. Prof. H. E. Clifford, of the Harvard Engineering School, who has won fame among his professional associates as a poet as well as an engineer, acted as toastmaster. The after-dinner speakers were Guy E. Tripp, head of the Westinghouse Electric Co., who discussed the "Five Brothers" bills pending in Congress for the control of the business of interstate corporations, Dr. A. E. Kennelly, who described the work accomplished by the International Electro-Technical Commission; C. O. Mailloux, President of the American Institute of Electrical Engineers; James Hartness, President of the American Society of Mechanical Engineers, and Prof. M. W. Franklin. Some 250 engineers were present at the dinner, all that the banquet hall of the City Club would accommodate.

One tangible result of this movement for coöperation and united action among the different engineering organizations in Boston, was the organization, two years ago, of the Engineers' Club, of Boston. An intangible result of probably even greater importance is the fostering of the spirit of coöperation and good fellowship among engineers engaged in all the different branches of the profession. The success which has followed the coöperative movement among Boston engineers should encourage similar action among the engineers of cities where such action has not been taken. Such a movement has already been inaugurated in a number of cities, and has been accompanied in places with the establishment of an Engineers' Club, with the usual benefits of a social club.

FIG. 2. HOLE IN 23RD ST. CAUSED BY GAS EXPLOSIONS
(View facing North River; asphalt pavement in foreground is on Fifth Ave. Paving on 23d St. is wood block.)

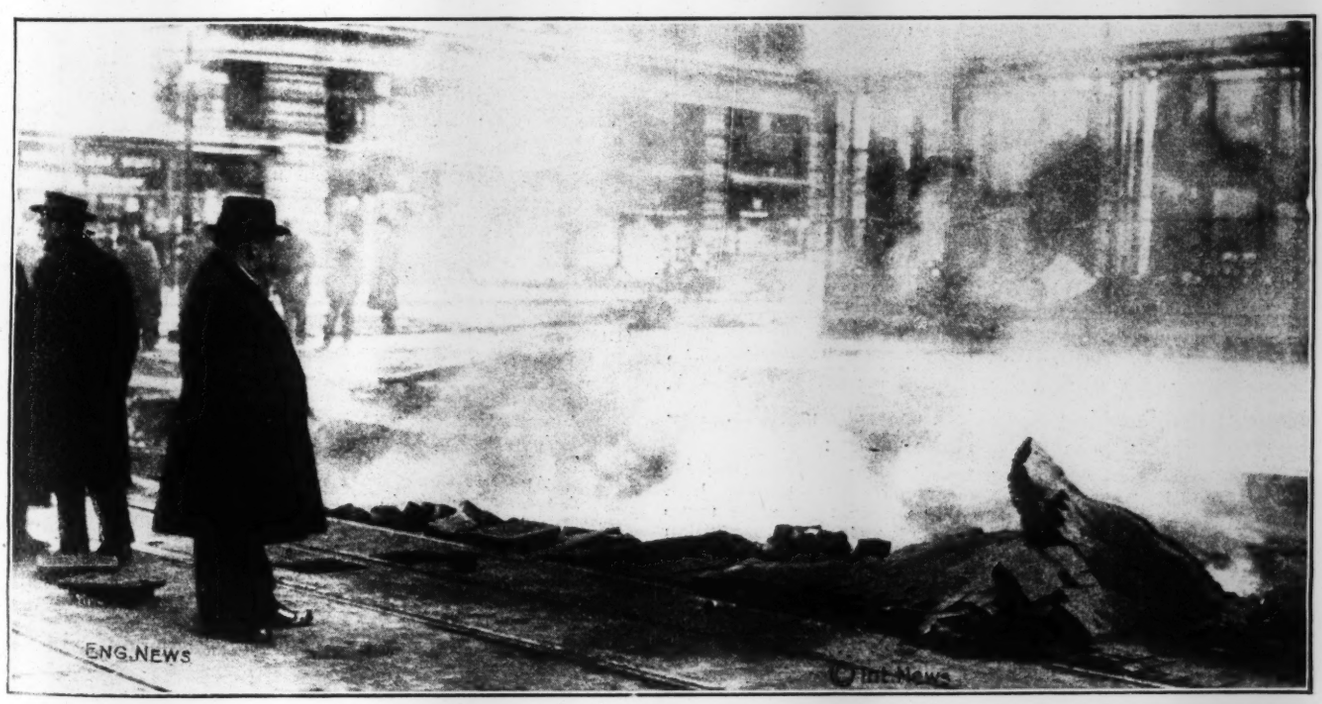


FIG. 3. DAMAGE TO WOOD-BLOCK PAVEMENT OF 23RD ST.
(View facing downtown portion of Fifth Ave.; Flatiron Building shown at upper left; windows of jewelry store opposite were not damaged by explosions.)

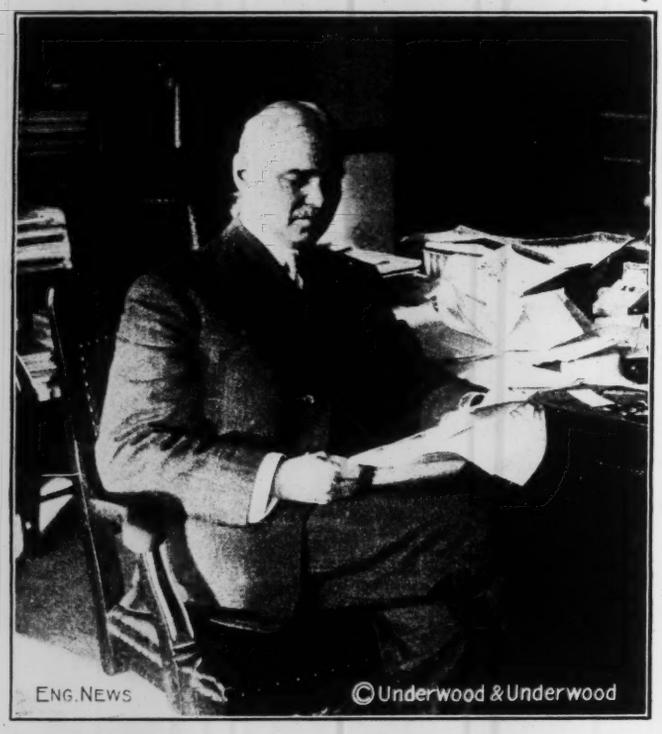
High Honors to an American Engineer

When Col. George Washington Goethals returns to Panama after his present visit to Washington and New York City, he can carry with him at least three gold medals, acquired within as many days, and enough laudatory addresses and speeches by eminent public men to fill a fair-sized volume.

The first medal above referred to bore this inscription:

This medal of the National Geographic Society is awarded to George Washington Goethals, to whose ability and patriotism the world owes the construction of the Panama Canal, Mar. 3, 1914.

The presentation was made at the annual dinner of the Society by the President of the United States, the Secretary of State acting as toastmaster. The guests included members of the diplomatic corps, cabinet members, prominent statesmen and army officers. In presenting the medal, President Wilson said many things complimentary of the engineering profession, as well as of Col. Goethals. Extracts from his remarks follow:



COL. GEORGE WASHINGTON GOETHALS, CHIEF ENGINEER OF THE PANAMA CANAL AT HIS OFFICE IN CULEBRA, C. Z., PANAMA

I am here to do what I suppose is an unusual thing for a society of this sort. It generally confers its honors upon those who have disclosed geography rather than upon those who have altered it. It is a sort of advertiser and custodian of the globe, but it is now about to honor a gentleman who has had the audacity to change the globe.

The engineering profession is one of the few creative professions. Those of us who have attempted to be literary men conceive that we have created conceptions of the mind, but we never can produce them in court. They are never visibly upon exhibition. But the magic of the engineer is that he can change the face of nature and show the work of his hands and that it is in some deep sense creative in character. The life of mankind on the globe is altered, for example, by the cutting and the use of the Panama Canal.

We have, therefore, to honor tonight the greatest living representative of this extraordinary profession. It seems to me to be natural, if I may say so with apologies to some of our friends present, that the greatest engineer should come from the United States. The United States has made the world very uncomfortable, but it has at least done so by the exercise of extraordinary dynamic qualities. It is not one of the statical nations of the world. It is one of the nations which has disturbed equilibrium, which has cut new paths for the thought and action of mankind.

And now there is to be elevated, and kept always on high, this new name upon which men are to enter the roads of new experience, a name which will not be blotted out until and unless the whole civilization of the world should change, the name of Colonel Goethals. The government of the United States lent him to the world and he has done this thing for the world. For it is our proud boast that we have cut this highway for all the sea-going ships of the world.

I take it for granted that we do not tonight forget that distinguished group of men who have been associated with Colonel Goethals—that gallant and devoted soldier who gave his very life to see that the great work was done at Culebra Cut; that man who made so much of this work possible—Surgeon-General Gorgas, by knowing how to hold disease off at arm's length while these men were given leave to work; Colonel Sibert, who built the walls of Gatun dam and created Gatun Lake, making it look to the eyes of the beholder as if nature had done the work over while he himself presided, and Colonel Hodges, who made the locks and the machinery by which these great things are administered.

But we are merely acknowledging the presiding character and genius which drew all the elements of this work together, which made it a work done by colaborers, not by rivals—work done as if it were the conception of a single mind and work done in the spirit of service as self-effacement which belongs to a great service of a great government.

There is nothing selfish in the eminence of Colonel Goethals. It is representative of a great profession; it is representative of a great government; it is representative of a great spirit.

I am glad that this thing was not done by private enterprise and that there is no thought of private profit anywhere in it, but that a government put itself at the service of the world and used a great man to do a great thing. That is the ideal of the modern world, that the services to mankind shall be commonly shared.

In his reply to President Wilson, Col. Goethals said: "It is easier to build canals than to find fitting words to express my gratitude," and he emphasized the idea that in accepting the medal he did so in the name of every member of the canal staff.

On the following evening, in New York City, Col. Goethals was presented with the first gold medal of the National Civic Forum for "distinguished public service." Carnegie Hall, one of the largest in New York City, was filled to overflowing, and many laudatory speeches were made by distinguished men. The following poem was read by the author, Percy MacKaye:

GOETHALS

By PERCY MacKAYE

(Delivered by the author at the National Testimonial to Col. George W. Goethals at Carnegie Hall, New York City, Mar. 4, 1914.)

A man went down to Panama
  Where many a man had died
To slit the sliding mountains
  And lift the eternal tide:
A man stood up in Panama
  And the mountains stood aside.

The Power that wrought the tide and peak
  Wrought mightier the seer:
For the One who made the isthmus
  He made the engineer,
And the good God He made Goethals
  To cleave the hemisphere.

The reek of fevered ages rose
  From poisoned jungle and strand,
Where the crumbling wrecks of failure
  Lay sunk in the torrid sand—
Derelicts of old desperate hopes
  And venal contraband;

MEDAL OF THE CIVIC FORUM AWARDED GEORGE WASHINGTON GOETHALS, CARNEGIE HALL, NEW YORK CITY, MAR. 4, 1914. DESIGNED BY PAUL H. MANSHIP

Till a mind glowed white through the yellow mist
And purged the poison-mold,
And the wrecks rose up in labor,
And the fevers' knell was tolled,
And the keen mind cut the world-divide,
Untarnished by world gold:

For a poet wrought in Panama
With a continent for his theme,
And he wrote with flood and fire
To forge a planet's dream,
And the derricks rang his dithyrambs
And his stanzas roared in steam

But the poet's mind it is not his
Alone, but a million men's:
Far visions of lonely dreamers
Meet there as in a lens,
And lightnings, pent by stormy time,
Leap through, with flame intense:

So from our age three giants loom
To vouch man's venturous soul:
Amundsen on his ice-peak,
And Peary from his pole,
And midway, where the oceans meet,
Goethals—beside **his** goal:

Where old Balboa bent his gaze
He leads the liners through,
And the Horn that tossed Magellan
Bellows a far halloo,
For where the navies never sailed
Steamed Goethals and his crew;

So nevermore the tropic routes
Need poleward warp and veer,
But on through the Gates of Goethals
The steady keels shall steer,
Where the tribes of man are led toward peace
By the prophet-engineer.

On Mar. 5, Col. Goethals was the guest of honor of the Economic Club, of New York, at a dinner at the Hotel Astor, attended by over a thousand persons and was the recipient of the medal of the National Institute of Social Science, which was presented by Mr. Hamilton W. Mabie.

Annual Meeting of the Indiana Sanitary and Water Supply Association

Reported by H. E. JORDAN*

The seventh annual meeting of the Indiana Sanitary and Water Supply Association was held at Indianapolis, Feb. 26 and 27, 1914. Aside from the program of addresses and papers, there was a banquet, with W. E. Stone, President of Purdue University, and Wm. L. Bryan, President of Indiana University, as speakers, and a trip to the Indianapolis Sewage Experiment Station. about 250 water-works men and health officers were in attendance. A feature of the meeting was an address on the commission-manager plan of city government, by Henry M. Waite, city manager, Dayton, Ohio [parts of this paper will appear in a later issue.]

Each vice-president of the Association is chairman of a committee on certain phases of water-works, and all papers and addresses were grouped under committee reports.

WATER-WORKS MANAGEMENT—The report of Howard A. Dill (superintendent, Richmond water-works) discussed the standard classification of water-works accounts established by the Indiana Public Service Commission.

Thomas Duncan, chairman of the commission, stated that in valuations for the commission, pavements laid after mains were installed, and not actually paid for by the utility, will not be allowed as an element of value in the going plant; that early losses incurred on bringing a system to an established business will be allowed as an item of going value, and that an important duty of the commission consists in not alone setting a fair valuation on the utility in question, but also in the regulation of

*Superintendent of Filtration, Indianapolis Water Co., Indianapolis, Ind.

sources of income, so that various classes of service will bear a just proportion of the expense.

John W. Alvord, in a paper on "The Fundamental Principles of Public Utility Valuations," traced the development of valuation work from 1895-1900, when a number of franchises in the Middle West began to expire, to the present time, when approximately $1,000,000,000 worth of property is being valued yearly.

LEGISLATION AND LEGAL DECISIONS—H. E. Barnard (chemist, Indiana State Board of Health, Indianapolis), in reporting for the committee on this subject, traced the change that has taken place since 1890 in the attitude of the courts in fixing the responsibility upon the municipality or corporation for furnishing impure water-supplies. Beginning with a case against the Plymouth (Penn.) Water Co., in 1891, when it is held that since the company did not know of the specific infection of its watershed, and the superintendent made occasional surveys of the stream, negligence of the company was not shown, the history was traced through the case against the Ashland (Wis.) Water Co., in 1898, where it was held that since the impure character of the water was a matter of common knowledge, no responsibility entailed upon the company, to the Mankato (Minn.) case in 1910, where it was held that the city operating the water plant in its corporate capacity was liable for damages to citizens whose health was injured by the use of the polluted water.

WATER-WORKS DEVICES AND MACHINERY—E. L. Loomis (superintendent, Valparaiso Home Water Co.) reported for this committee. He detailed the experience of this company in the use of various types of pumps. A high-duty pump in service will deliver 1,000,000 gal. of water at a coal cost of $2.98; another, noncompound crank and flywheel type, at $4.14, and a third, a duplex pump, at $6.26.

Ellwood Ramsey, of the Baker-Vawter Audit Co., in discussing "Elimination of Water Waste," cited the case of a town in Indiana which applied to the State Board of Health for aid in developing a new supply. It was found that this town was using over 500 gal. per capita. In another city 1,500,000 gal. daily were found to be passing into a factory supply system without being paid for. A company has equipped all its inspectors with aquaphones, with instructions to examine every service for ground and house leaks.

WATER PURIFICATION—The report for this committee was submitted by H. G. Morgan (city sanitarian, Indianapolis). Mr. Morgan stated that a survey of typhoid cases in Indianapolis, in 1913, showed 90% occurring where privy vaults were used, and 86% using water supplied by private wells, while a zone epidemic of 42 cases was traced to a carrier employed in a dairy.

C. N. Hilliard (professor of sanitary science, at Purdue University) in a paper "Water-Borne Diseases, the Pulse of a Community," detailed the urban and rural death rate from typhoid in Indiana as follows:

	Population		Typhoid death rate per 100,000	
	Urban	Rural	Urban	Rural
1911	608,511	321,643	25.7	23.6
1912	625,870	330,341	24.6	22.8
1913	641,175	338,032	27.9	26.6

Allan J. McLaughlin, Surgeon, United States Public Health Service, in a paper on "The Relation of Polluted Water-Supplies to Mortality of Children," called attention to the winter enteritis in the cities using water-supplies along the Great Lakes. In conclusion he said:

> In cities with polluted water-supplies, the rate for diarrhœa and enteritis under two years is abnormally high in the winter and spring months, and the substitution of a safe for a polluted public water-supply results in a marked reduction of deaths of infants from the above named diseases, especially during the months from December to June.

In discussing the above papers, H. E. Jordan (Indianapolis Water Co.) stated that the death rates in Indianapolis following water purification had not followed the Mills-Reincke phenomenon. During the 22 years from 1891-1912, inclusive, the average typhoid rate was 42.8 per 100,000—51.8 before filtration and 27.1 after—while the percentage of total deaths due to typhoid was 3.1 before filtration and 1.8 after filtration. In spite of this, the general death rate was reduced only from 15.3 to 14.8 per 1000, and the rate from diarrhœal diseases from 83.8 to 79.9 per 100,000. The conclusion is that water-supply was a minor factor in the typhoid situation in Indianapolis.

In a paper on "The Volume of Chemical Change During Water Purification," C. K. Calvert (chemist, Indianapolis Water Co.) presented some interesting figures on the amounts of chemical substances carried by a number of rivers in the United States, including the White River, at Indianapolis. Gains and losses due to the operation of the Indianapolis filters were also given. Assuming for the White River an annual average flow of 38,000,000,000 cu.ft. or 12,000,000 tons from 1240 sq.mi. of drainage area, then the river carries 434,400 tons of total solids per year, or enough to lower the whole drainage area 0.003 in. per year or 1 ft. in 4000 years. Estimating the bacterial content of the water at 320 billions per pound, then 2522 tons of living bacteria or 2 tons per sq. mi. of drainage area pass the sampling point each year.

FLOODS AND FLOOD PREVENTION—Morris Knowles, engineer member of the Pittsburgh Flood Commission, and Major J. C. Oakes, of the Board of Officers on River Floods, discussed the subject of "Flood Prevention" with particular reference to the floods of 1913.

Reservoirs on the headwaters of streams, to be used also as sources of power and to furnish water during dry seasons, with joint federal and state control, were advocated by Mr. Knowles, while Major Oakes decried their efficiency, claiming in the case of the Wabash (Ind.) River flood of 1913, that to have kept it within its banks from Mar. 25 to Apr. 21, would have required reservoir capacity of 260,000,000,000 cu.ft. (930 sq.mi., 10 ft. deep).

Particular criticism was directed to the building up of flood plains of rivers in their flow through cities and encroachments by bridges, railroad embankments and business structures upon the dry weather stream beds.

John W. Alvord, in discussing these addresses, claimed that each city's problems must be worked out according to the local conditions, giving as examples the flood-prevention plans for Columbus and Dayton, Ohio. In the former, channel improvements were advised, both on ground of safety and first cost, while at Dayton this method was too expensive—but reservoir sites were found capable of being developed at feasible cost that would control greater floods than those of 1913.

OTHER PAPERS—R. C. Wheeler presented a paper on the sewerage system and sewage-treatment works at Vincennes, Ind. [Abstracted on p. 534, of ENG. NEWS, of

Mar. 5, 1914.—Ed.] E. R. Cummings (professor of geology at Indiana University, Bloomington) detailed the characteristics of the geology of Indiana as regards water-supply. The location of reservoirs on the knobstone formation was advised, so far as this is feasible, on account of the fact that the rock named is impermeable and insoluble.

Business Proceedings—A committee was appointed at the opening session to consider affiliation with the American Water Works Association. The committee's report was favorable, but in view of the fact that some doubt was raised as to the possibility of retaining the interest of the State Board of Health and county health officers, it was decided to continue as an independent State organization for the time.

For President of the Association, H. A. Dill (President Richmond Water Co.) was chosen to succeed Dow R. Gwinn, of Terre Haute. For Secretary-Treasurer, W. F. King (Assistant Secretary, State Board of Health, Indianapolis) was continued in office.

Vice-Presidents, each to be chairman of a committee, as stated, were elected as follows:

C. H. Hurd (vice-president, Indianapolis Water Co.), Water-Works Management and Accounts.

H. E. Barnard (state food and drug commissioner, Indianapolis), Legislation and Legal Decisions.

R. L. Sackett (professor of hydraulic and sanitary engineering, Purdue University, Lafayette), Stream Pollution and Sewage Disposal.

W. H. Durbin (superintendent, Evansville Water Department), Water Purification.

Quince Walling (superintendent, Muncie Water Co.), Water-Works Devices and Machinery.

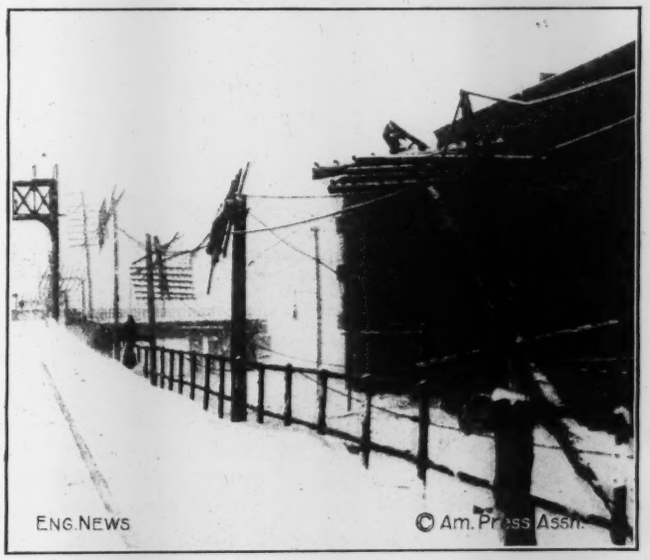
Natural vs. Designed Poles During a Blizzard

The snow which fell in the blizzard that struck New York City and vicinity on Mar. 1-2 was very damp, so that it soon weighted down all suspended wires with a coating sometimes as thick as 4 in. in diameter. After this coating had become partly or wholly frozen there was a very high gale, registered on the Weather Bureau anemometers at a maximum of 84 miles per hour. These conditions combined to cause great damage to all of the overhead wires in the vicinity of New York City, but particularly along the railways in New Jersey, where the combination of wind and snow seemed to have been greatest.

It so happens that the service lines along all of the railways in this territory are of the old wooden-pole type, with the exception of those along the Hackensack Meadows Division of the Pennsylvania R.R., where the telegraph line is carried on reinforced-concrete poles and, on the other side of the track, a power-transmission line on structural-steel poles. These two designed pole lines were the only ones crossing the Meadows which did not suffer severe breakdowns.

The accompanying view is taken near the Erie R.R. terminal and shows a typical condition along the wooden-pole line. Such breakage was common along the Lackawanna, Erie and Jersey Central roads, where the wooden poles were used. All of these lines are of long standing and are not standardized as to size of poles and span; but in general, spans of 120 ft. are used and the wires carried number from 50 to 100. The breaks are either in the crossarms, or in the pole, just below the crossarms, or at midheight.. Very few poles are uprooted, but there are some cases where the wires are stripped from the insulators and the pole left standing. As would be expected, the break of one pole has always started a train of breaks. Anchor poles would have helped to localize damage.

On the line of the Pennsylvania, the contrast is quite marked. The pole lines there, which were placed in 1909, suffered practically no damage. In a few cases, the crossarms were broken or pulled from the concrete poles, but no damage was noticed to any one of the 202 concrete or 77 steel poles crossing the Meadows.



Wooden Telegraph Poles near Erie R.R. Terminal, Jersey City, after Blizzard of March 1-2.

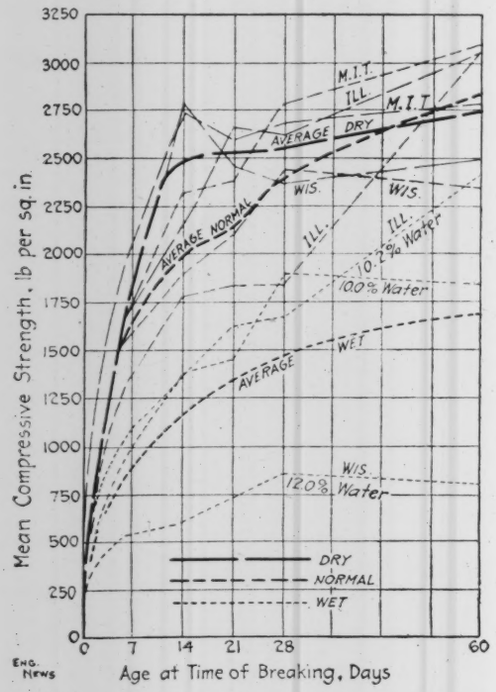
The concrete poles are from 25 to 50 ft. in height above ground and are embedded in timber grillages below which the pole projects into the soft earth. They are square in cross-section and reinforced with rods in each corner, to resist a pull of 6500 lb. at 6.5 ft. below the top. The general span across the Meadows is 120 ft. They carry about 40 wires and two cables at present. Special guys are provided along curves and at crossings.

The transmission-line poles are for 300-ft. spans; they are from 50 to 70 ft. in height and carry some 16 wires and cables. Each pole is of latticed structural steel, square in section with one angle at each corner and single-angle bracing. They are set in heavy concrete bases on timber piles.

Some Tests on Strength of Overwet Concrete

The table and diagram on the next page show plainly the deleterious effect of too much water in concrete. The data were compiled by the Committee on Specifications and Methods of Tests for Concrete Materials, of the American Concrete Institute, in connection with an extended investigation into a standardization of concrete test pieces. While the tests noted here were made in an effort to arrive at a proper proportion of water to use in mixing test pieces, they at the same time show definitely

the effect of a variation in water content which must be of similar, although not necessarily proportionate, importance in fieldwork.



TIME-STRENGTH CURVE OF CONCRETES MIXED WITH DIFFERENT DEGREES OF WATER

The tests reported were made under the same prescribed conditions of standardization and manipulation, but with necessary local variations in material, by three college laboratories. The concrete was approximately a 1:2:4 mix, with coarse aggregate of from ½ to ¾ in. in diameter. Each value given is an average of four 6-in. diam. by 6-in. cylinders. The amount of water used varied with conditions of sand and gravel, but the consistency is described as follows in the Committee's report:

Because of the difference in the effect of different sands upon the consistency, it was impossible to specify a definite percentage of water. Tests made by members of the committee indicate that the most uniform degree of consistency can be obtained by adopting the Chapman consistency test, which consists in filling a slightly tapering cylindrical form with concrete, immediately inverting this, and by repeated trials finding the amount of water which will cause the concrete to just begin to slump when the form is removed. A dry mix is, of course, unsatisfactory, while it is almost impossible to describe a very wet mix which will insure uniformity.

Speaking generally, however, the dry mix is about 8% water, the normal about 9% and the wet 10%. These percentages, it will be appreciated, are much lower than those required to give similar consistencies in actual concrete work, but the comparison is the same.

The accompanying table gives the actual compressive values up to 60 days and the curve the average of the three laboratories for the three degrees of consistency.

TABLE SHOWING EFFECT OF WATER PERCENTAGE ON COMPRESSIVE STRENGTH OF CONCRETE 6x6-IN. CYLINDERS

	Average compressive strength, lb. per sq.in.								
	University of Illinois			University of Wisconsin			Mass. Institute of Technology		
Age	Dry	Normal	Wet*	Dry	Normal	Wet†	Dry	Normal	Wet‡
7 days...	1751	1390	1103	1690	1580	533	2047	1740	965
14 days...	2140	1775	1354	2795	1905	600	2742	2320	1372
21 days...	2658	1816	1623	2450	2095	722	2594	2396	1464
28 days...	2615	1820	1657	2380	2430	860	2679	2882	1895
2 mo.....	3056	3063	2410	2485	2340	787	2761	3092	1830

* 10.2% water. † 12.0% water. ‡ 10.0% water.

NEWS NOTES

Iceland's First Railroad is soon to be commenced according to a press dispatch dated Mar. 2.

A Prize of $2500 for the best plan submitted for the reconstruction of the city of Dublin, Ireland, has been offered by Lord Aberdeen, according to the March issue of the "Journal" of the American Institute of Architects. Further information can be had from the Local Government Board at Dublin.

A Severe Fire in St. Louis, Mo., destroyed the seven-story red-brick clubhouse of the Missouri Athletic Club, at 2 a.m. Mar. 9. Of the 100 guests entrapped on the top two floors, ten bodies have been recovered, about 22 are missing and 17 were injured in making their escape. The ground floor of the building was shared by the Boatmans' Bank. The cause of the fire, which burned for 16 hours, has not been determined.

A 12-in. Coast Artillery Gun Exploded during test at Sandy Hook on Mar. 9. Press reports state that the breech of the gun was hurled to the rear a quarter of a mile, landing on the lawn of the officers' club, and that a 100-lb. fragment of steel was driven half way through the wall of the concrete bomb-proof, which sheltered the officers of the testing squad. The officer in charge of the firing, who was stationed on the observation bridge, was injured by a piece of flying metal.

The Railway Educational Bureau, of the Union Pacific and Illinois Central R.R.'s, whose work was described in "Engineering News," Oct. 7, 1909, has arranged to conduct its courses so that officials and employees of other railroads can participate in these benefits. It is reported that 25 courses of instruction are now available and that all the privileges of the bureau may be secured by any subscriber for a moderate fee. The headquarters of the Bureau is at Omaha, Neb.

A Comprehensive System of Subways for Providence, R. I., has been recommended to the City Council by a special committee of that body. The recommendations are based on the findings of W. W. Lewis, Assistant Engineer of the Boston Transit Commission. There would be, by this scheme, four subway lines for surface cars radiating from a central transfer station at Exchange Place to the Seekonk River, to North Burial Ground, to Trinity Sq., and Olneyville, aggregating about 3¼ miles of line. The work would be completed by 1923 at a cost of $13,600,000. After construction, they would be leased to the Rhode Island Co. (which operates the surface-car system) for 4 or 4½% annual charge.

The Regular Spring Crop of Small Dam Failures has commenced. Clippings reaching this office this week show failures on Feb. 26, at Redlands, Calif., on Mar. 2, at Westport, Conn., and Potsdam, N. Y., and on Mar. 3, at Rochester, N. H., and Canterbury, Conn. None of the dams was of sufficient size to cause great damage to downstream property, but the news of their failures is significant as an indication of the very general insecurity of many such structures under the severe stress of spring floods.

The Tank Steamer "Oklahoma," which Broke in Two on Jan. 4, with a loss of 26 lives, has just been reported structurally weak by the Government inspectors—John F. Blair and Andrew J. Savage—appointed to investigate the accident. The report absolves the ship's officers of any responsibility, and states that there was nothing to show that the steamboat inspection service was dilatory in its duties or supervision over the vessel. Furthermore, it states that there is no reason to think that the designers, builders, or owners of the "Oklahoma" were wilfully responsible for the structural weakness of the vessel. This accident was discussed editorially in "Engineering News," of Jan. 15, p. 136.

The Causes of Forest Fires, and the relative importance of these causes, in all the national forests of the United States were recently stated by the Bureau of Forestry, Washington, D. C., to be as follows:

	No. fires	Per cent.
Lightning	1571	34.76
Unknown	886	19.60
Railroad locomotives	543	12.01
Campers	513	11.35
Incendiarism	452	10.00
Miscellaneous	257	5.69
Brush burning	232	5.13
Sawmills, donkey engines, etc	66	1.46

Prize for Highway Engineering Students—The Barber Asphalt Paving Co. has offered a prize of $100 for the best paper written by a member of the graduating classes of any one of the leading engineering schools. The title suggested is "Asphaltic Materials for Highway Construction." The paper and its conclusions may be based upon service tests and the lessons of experience; the physical qualities or chemistry of asphalt; or it may combine any two of these lines of in-

vestigation. The length of the paper is limited to 3000 words and all manuscripts must be received not later than June 1, 1914.

A Day's Work at Snow Removal in New York City—Twenty thousand shovelers and truck drivers removed 121,386 cu.yd. of snow in Manhattan, the Bronx and Brooklyn, on March 3. Street Cleaning Commissioner Fetherston and Snow Removal Superintendent Gunther, compiled the following statistics of the day's work:

Borough	Labor	Trucks	Trips	Cubic yards removed
Manhattan	4,663	1,632	16,000	80,000
Bronx	635	208	2,008	8,032
Brooklyn	2,000	831	7,412	33,354
Totals	7,296	2,671	25,420	121,386

In addition to the men above tabulated, there were 6708 regular employees of the Street Cleaning Department and 3523 emergency men put on by the Department, 3500 refuse collectors, and 250 men of other departments (e.g., the Park Department), removing snow.

The Human Reaction of a Water Main Break is seldom put on record. The water-works fraternity is indebted to the Springfield (Mass.) "Republican" for the following graphical account of such a reaction, related early in the morning of March 1 by a woman who, with the other members of several families, had been driven out into the street by the undermining and wrecking of a house by the water escaping from a 36-in. water main under heavy pressure (see "Eng. News," March 5, 1914, p. 540). The woman said:

I was sleep, and wake up with thunder and shake. I poke my husband in his ribs quick, like that, and say, "Val, wake hurry; something happen." He tell me to shut up. I am scare bad. The house shake some more. I get up, grab my babies and start downstair. Then Val he wake up and come too. We wade in water while the poliss they wake up the rest. Ain't the house a worse though? There goes the dishes we got yesterday.

Proposed Amendments to the Constitution of the American Society of Civil Engineers Defeated—All three of the proposed amendments to the constitution of the American Society of Civil Engineers, abstracts of which were given in our issue of Jan. 29, p. 262, in the report of the annual meeting of the society, were defeated by letter ballot. The results, which were announced at the meeting held in New York City, Mar. 4, were as follows: Amendment A, for 1494, against 1628; Amendment B, for 1550, against 1612; Amendment C, for 1270, against 1826. Amendment A was the one proposed by Messrs. Keefer and others for redistricting the membership and had the support of those who were most strongly opposed to the so called Hovey amendments, one purpose of which was to change the status of the secretary. The members were circularized by both sides, those in favor of Amendment A and opposing Amendments B and C being signed by J. Waldo Smith, Alfred Noble and Allen Hazen, and that opposing Amendment A and favoring Amendments B and C signed by Elmer E. Corthell, William L. Darling, Mordecai T. Endicott, Desmond Fitzgerald, Henry W. Hodge, George F. Swain and 41 other members prominent both in the society and in the engineering world.

Two Interesting Trestle Accidents of recent date were caused by derailments during snow-removal operations. One was on the Keokuk and Centerville Division of the Chicago, Burlington & Quincy R.R., at Arbela, Mo., on Feb. 23. The derailment of a train while bucking snow a short distance from the trestle over Little Creek, stripped the deck off the trestle and knocked down two bents.

The other accident (briefly noted in "Engineering News," of Feb. 26, p. 487) was caused by the derailment of a snow plow on the Moncton & Buctouche Ry., near Moncton, N. B., on Feb. 20. E. G. Evans, General Manager of the railway, informs us that a special train consisting of a snow plow, two engines and a car, was proceeding toward Moncton at about 12 mi. per hr. About 150 ft. north of the wooden trestle which failed, the plow jumped the track and was almost upon the trestle before the head engineer noticed the derailment. He blew for brakes and applied his own, but the rear engineer who was killed evidently did not hear the signal, as the exhaust indicated that his engine was in operation until it fell over the trestle. About half way across, the plow toppled over the side, knocking the caps of the trestle out of place and carrying the train with it. When the caps of the trestle were dislodged, about 70 ft. the trestle collapsed. Four trainmen were killed and two injured.

A Remarkable Cut in Salaried Officers and Engineers in the service of the City of Boston has been made by Mayor Curley. Thirteen assistant engineers and draftsmen in the Department of Public Works were summarily suspended on Feb. 28 for a period of five months. The letter of the Mayor sent to Commissioner of Public Works Rourke was as follows:

You are hereby directed to abolish the position of consulting engineer, high-pressure service, which position is at present filled by Clarence Goldsmith at a salary of $3500 per annum; to abolish the position held by John H. Conry, district foreman of sewer service, East Boston, at $1800 per annum; the position of chief inspector of sewer service held by James J. Conway at $2500 per annum; the position of district engineer of sewer service held by Alexander L. Kidd at $2400 per annum.

You are hereby directed to grant leave of absence, without pay, for a period of five months to Executive Engineer Henry W. Sanborn, receiving a salary of $3000 per annum.

You are hereby directed to reduce the salary of Edgar S. Dorr, chief engineer of sewer service, from $3500 to $2500 per annum.

You are hereby directed to remove William J. Welch, superintendent distribution branch, Albany-street water yard, receiving a salary of $3000 per annum.

These changes, with the exception of William J. Welch, to become operative on March 7, and in the case of Mr. Welch, to become operative on April 1.

You are hereby directed to promote Edward F. Murphy from district engineer to engineer in charge of sewer service, and increase his salary from $2400 to $3500 per annum.

Most of those affected by the above order have been in the city's service for many years.

The lack of method in making suspensions is shown by the fact that, with one exception, the names were taken in order from the salary list of April, 1913, a printed document, and a new man, whose name had not been entered in the list, was retained. No consideration appears to have been given to length of service, efficiency, amount of salary or the actual need of the men to carry on the work of the department.

All of the men suspended are under the civil service, so that, while there appears to be an impression in Boston that a suspension is equivalent to a removel, legal action would probably result in at least a part of them being taken back if, at the end of the five months the Mayor should attempt to fill the positions with other appointees.

The total amount saved by the reduction in salaries above given is at the rate of $13,000 per annum. Still more sweeping and wholesale removals were made in the Fire Department and the Park and Recreation Department.

PERSONALS

Mr. W. J. Sadler has been appointed City Engineer of Port Townsend, Wash., at a salary of $1500 per annum.

Mr. Charles F. Barnhill has been appointed Master Mechanic of the Gulf, Colorado & Santa Fé Ry., at Silsbee, Tex.

Mr. H. C. McCurdy has been appointed City Engineer of Vernon, Calif., succeeding Mr. Benj. F. Dupuy, now City Engineer of Long Beach, Calif.

Mr. D. C. Wilson has been appointed Electrical Engineer of the Union Pacific R.R., with headquarters at Omaha, Neb., succeeding Mr. A. J. Collett, resigned.

Mr. F. W. Bettle, recently Assistant Engineer of the Terminal R.R. Association of St. Louis, Mo., is now Division Engineer of the Texas & Pacific Ry., at New Orleans, La.

Mr. L. H. Phetteplace, General Superintendent of the Carolina, Clinchfield & Ohio Ry., has been promoted to be General Manager. He will retain his headquarters at Erwin, Tenn.

Mr. W. H. Davies, Superintendent of Air Brakes of the Chicago & Alton R.R., has resigned to accept a similar position with the Oregon Short Line R.R., at Salt Lake City, Utah.

Mr. C. S. Heritage, Supervisor of the Washington Terminal Co., Washington, D. C., has been promoted to be Engineer, succeeding Mr. C. H. Spencer, resigned, as noted elsewhere.

Mr. E. H. Lee, M. Am. Soc. C. E., Chief Engineer of the Chicago & Western Indiana R.R. and the Belt Ry. of Chicago, Ill., has been elected Vice-President in addition to his other duties.

Mr. Harry E. Funk, formerly Assistant Engineer of the Brooklyn Rapid Transit Co., assigned to building construction, has been appointed Superintendent of Buildings of the company.

Mr. Henry Tschetschott, Professor of mining engineering at the St. Petersburg Mining Institute, St. Petersburg, Russia, is taking a special course at the Massachusetts Institute of Technology.

Mr. J. H. Roach, Assoc. M. Am. Soc. C. E., Assistant Engineer of Construction of the Lake Shore & Michigan Southern Ry., Cleveland, Ohio, has been appointed Valuation Engineer of the company.

Mr. F. L. Burckhalter, District Engineer of the Southern Pacific Ry., at Portland, Ore., has been promoted to be Division Superintendent with headquarters at Portland, Ore., succeeding Mr. L. R. Fields.

Mr. W. S. Hawley, Assoc. M. Am. Soc. C. E., Division Engineer of the New Orleans Great Northern R.R., at Bogalusa, La., has been promoted to be Chief Engineer, and the office of Division Engineer has been abolished.

Mr. Carl R. Gray, Jr., son of Mr. Carl R. Gray, President of the Great Northern Ry., has been appointed Special Assistant to the General Superintendent of the First district of the St. Louis & San Francisco R.R., at Springfield, Mo.

Mr. Arthur Maguire, recently Division Engineer of the San Pedro, Los Angeles & Salt Lake R.R., at Los Angeles, Calif., has been promoted to be Chief Engineer, succeeding Mr. E. G. Tilton, resigned as noted in our issue of Feb. 12.

Mr. Charles J. McIntosh, formerly with the J. J. Case Threshing Machine Co., of Racine, Wis., has been elected President of the Modern Steel Structural Co., Waukesha, Wis., succeeding Mr. Samuel B. Harding, resigned, as noted elsewhere.

Mr. J. C. Auten, recently Division Engineer of the Philadelphia, Baltimore & Washington R.R., at Wilmington, Del., has been promoted to be Principal Assistant Engineer, succeeding Mr. C. I. Leiper, promoted, as noted in our issue of last week.

Mr. R. E. Palmer, M. Can. Soc. C. E., Chief Mining Engineer of the Rio Tinto Mines, in Spain, has been appointed Consulting Engineer of the company, with offices in London, England, where he will also act in a consulting capacity for several other companies.

Mr. E. L. Crugar, Assistant Chief Engineer of the Chicago & Alton R.R., has resigned and the office has been abolished. Messrs. I. C. Brower, C. L. Hartwell and C. A. Stelle, Division Engineers, have also resigned, and the office of Division Engineer has been abolished.

Mr. George P. Robinson, for several years connected with the city engineering department of Los Angeles, Calif., as Assistant and Supervising Engineer in charge of paving and storm-sewer improvement work, has been appointed City Engineer of Santa Barbara, Calif.

Mr. George H. Harris, Assoc. Am. Inst. E. E., recently Superintendent of the Railway Department of the Birmingham Railway, Light & Power Co., Birmingham, Ala., has been appointed General Superintendent of the San Francisco-Oakland Terminal Ry., San Francisco, Calif.

Mr. W. E. Guignon, recently Assistant Engineer, Northwest system, Pennsylvania R.R. Lines West of Pittsburgh, Chicago, Ill., has been promoted to be Division Engineer of the Zanesville division of the Central system, at Zanesville, Ohio, succeeding Mr. A. C. Watson, transferred.

Mr. J. M. Barber, Jun. Am. Soc. C. E., recently Designing Engineer with Edward A. Tucker & Co., Engineers and Contractors, Boston, Mass., has been appointed Instructor in civil engineering at the Massachusetts Institute of Technology. Mr. Barber graduated from the Institute in 1907.

Mr. George A. Gallinger, of Pittsburgh, Penn., has been placed in charge of the pneumatic tool department of the Ingersoll-Rand Co., New York City, with the title of Manager of Pneumatic Tool Sales. Mr. Gallinger is familiar with the mechanical as well as the commercial part of the business.

Messrs. J. P. Corcoran, M. Donahoe and H. Kleine, formerly Supervisors of the Chicago & Alton R.R., have been promoted to be Roadmasters, a newly created office, in charge of maintenance-of-way, succeeding Assistant Chief Engineer E. L. Crugar, resigned as noted elsewhere, and three Division Engineers resigned.

Mr. S. C. Hulse, Assoc. M. Am. Soc. C. E., formerly Principal Assistant Engineer, Compañia Agricola y de Fuerza Electrica del Rio Conchos, S. A., Santa Rosalia, Chihuahua, Mex., sailed from New York, Mar. 3, for Nicaragua where he is to report on a power development project for the Canadian Agency, 1 Broadway, New York City.

Mr. F. W. Hoover, General Manager of the Tennessee Power Co., has been appointed executive officer of the Chattanooga (Tenn.) Railway & Light Co., of which he is Vice-President. He will also continue his work as General Manager of the Tennessee Power Co., with offices in the Hamilton National Bank Bldg., Chattanooga.

Mr. C. A. Brandt, Mechanical Engineer of the Cleveland, Cincinnati, Chicago & St. Louis Ry., at Beech Grove, Ind., has been appointed Assistant Master Mechanic of the Chicago, Michigan and the Peoria & Eastern divisions, with headquarters at Indianapolis, Ind. Mr. Brandt is succeeded as Mechanical Engineer by Mr. W. E. Ricketson.

Mr. Louis W. Hill, Chairman of the Board of Directors of the Great Northern Ry., has been reëlected President of the company to succeed Mr. Carl R. Gray, whose resignation to become President of the Western Maryland Ry. was noted in our issue of Feb. 26. Mr. Hill, who is a son of Mr. James J. Hill, was President from 1907 to July 1, 1912.

Mr. J. B. Phinney, for many years engaged in railway and municipal work in Ohio and neighboring states, is now engaged in private practice as Surveyor and Civil Engineer at Dade City, Fla. Mr. Phinney was recently Chief of Party on the drainage survey of the Florida Everglades under the supervision of the Everglades Engineering Commission.

Mr. George H. Binkley, M. Am. Soc. C. E., recently Chief Engineer of the Mesa Improvement Co., Chandler, Ariz., has been appointed Chief Engineer of Maintenance-of-Way of the San Francisco & Oakland Terminal Ry., Oakland, Calif., succeeding Mr. Edward M. Boggs, M. Am. Soc. C. E., who has been made Engineer in charge of wharves and docks.

Mr. O. P. M. Goss, Assoc. Am. Soc. C. E., Engineer in Forest Products, U. S. Forest Service, Seattle, Wash., in charge of the timber testing laboratory at the University of Washington since 1904, has been appointed Consulting Engineer of the West Coast Lumber Manufacturers Association and the Pacific Coast Creosoting Companies Association.

Mr. T. E. Keegan has been appointed Assistant Manager of the Tyler Underground Heating System, Pittsburgh, Penn. Mr. Keegan was formerly with the Atlas Engine Co., Indianapolis, Ind., the American Stoker Co., New York City, the Consolidated Engineering Co., Chicago, Ill., and for the last two years with the Open Coil Heater & Purifier Co., Indianapolis, Ind.

Mr. Frank W. Hanna, M. Am. Soc. C. E., recently Engineer, United States Reclamation Service, Washington, D. C., has been promoted to be Supervising Engineer of the Southwestern division of the Reclamation Service, with headquarters in the Wright & Callender Bldg., Los Angeles, Calif. This division includes all the projects in Southern California, Arizona, Nevada, Utah, Texas and New Mexico.

Mr. Samuel B. Harding, for the past 13 years President of the Modern Structural Steel Co., of Waukesha, Wis., has retired to become Consulting Engineer of the company. Mr. Harding was formerly Engineer with the Wisconsin Bridge & Iron Works, Milwaukee, Wis., and at the time this company was absorbed by the American Bridge Co. he resigned and founded the Modern Structural Steel Co.

Mr. Harry O. Cole, M. Am. Soc. C. E., whose resignation as Resident Engineer of the Pacific division of the Panama Canal was noted in our issue of Feb. 19, and Mr. Emmert L. Cole, Assoc. M. Am. Soc. C. E., formerly with the American Bridge Co. and the Carnegie Steel Co., Pittsburgh, Penn., announce that they have incorporated under the firm name of Cole Brothers, Inc., Engineers and Contractors, 16 St. Paul St., Baltimore, Md.

Mr. Earl A. Averill, Assoc. Am. Soc. M. E., has resigned as Managing Editor of the "Railway Age Gazette Mechanical Edition," to become Engineer of Operation of the Standard Stoker Co., Grand Central Terminal, New York City. Mr. Averill is a mechanical engineering graduate of Cornell University, class of 1900, and after four years' experience in the mechanical department of the Chicago, Burlington & Quincy R.R. began editorial work with the "Railway and Engineering Review," of Chicago, Ill. In 1906 he joined the staff of the "American Engineer and Railroad Journal," of which, in 1910, he was made Managing Editor. He continued in the same position when the paper was purchased by the "Railway Age Gazette."

Mr. C. H. Spencer, Engineer of the Washington Terminal Co., Washington, D. C., has been appointed Assistant District Engineer of the Eastern district, Division of Valuation, Interstate Commerce Commission, with headquarters at Washington, D. C. Mr. Spencer is a native of Detroit, Mich., and a graduate of the University of Michigan, class of 1896. He started engineering work as a rodman on the Lake Shore & Michigan Southern Ry. Subsequently he was Chief Draftsman and Engineer of Construction of the Baltimore & Ohio R.R. Mr. Spencer was Resident Engineer of the Baltimore & Ohio R.R. in charge of construction of the Washington, D. C. terminal, and since its completion has been Engineer of the Washington Terminal Co.

The Pennsylvania R.R. announces the following promotions and transfers: Mr. J. R. McGraw, formerly Division Engineer of the Buffalo division, has been made Division Engineer of the Maryland division; Mr. E. B. Wiseman, formerly Division Engineer of the Monongahela division, has been made Division Engineer of the Buffalo division; Mr. J. B. Hutchinson, Jr., formerly Division Engineer of the West Jersey & Seashore R.R., has been made Division Engineer of the Monongahela division; Mr. W. B. Thomson, formerly Assistant Engineer of the Elmira and Canandaigua divisions of the Northern Central Ry., has been promoted to be Division Engineer of the Williamsport and Susquehanna divisions; Mr. H. L. Thomas, formerly Supervisor of the Philadelphia division, has been promoted to be Division Engineer of the Elmira division.

Mr. Louis C. Hill, M. Am. Soc. C. E., Supervising Engineer, United States Reclamation Service, Los Angeles, Calif., has been promoted to be Consulting Engineer. Mr. Hill has been with the Reclamation Service since 1905 and was formerly stationed at Phoenix, Ariz. He is a graduate of the University of Michigan, class of 1886, and his first engineering experience was with the Duluth, Red Wing & Southern R.R as Assistant Engineer in 1887-'88. Later he was with the Great Northern Ry. In 1889 he returned to the University of Michigan and graduated in 1890 in electrical engineering. For 13 years he was Professor of hydraulics and electrical engineering at the Colorado School of Mines. In 1903 he was appointed to a position in the United States Geological Survey and upon the establishment of the United States Reclamation Service was made Supervising Engineer in charge of all work in Arizona, New Mexico, Texas, Utah and southern California. Mr. Hill will probably retain his headquarters in Los Angeles. He is succeeded as Supervising Engineer by Mr. Frank W. Hanna, M. Am. Soc. C. E., recently Engineer of the Reclamation Service at Washington, D. C., as noted elsewhere in these columns.

Sir William Willcocks, the famous English Irrigation Engineer, intends to visit the United States for the first time in April. He is to deliver an illustrated address on "River Regulation and Control in Antiquity" at the annual meeting of the National Drainage Congress, to be held at Savannah, Ga., Apr. 22-25. There is probably no person in the world better qualified to talk on the subject selected than Sir William Willcocks. He was born in India and received his technical education at the Thomason Civil Engineering College, at Roorkee, India. He laid the foundations of his world-wide reputation as an irrigation engineer by his labors in the public-works service in India, where he served for 11 years. In 1883 he went to Egypt, where he became one of the Assistants of Sir Colin Scott-Moncrieff and a pioneer in the rehabilitation of the Nile irrigation systems. For many years he was connected with this work and achieved international fame as the Engineer in charge of the design and construction of the Assuam dam. Later he was Manager of the Cairo water-works. From 1908 to 1909 he was Engineer Advisor to the Turkish Government and had charge of the preparation of the plans for reclaiming the famous delta of the Euphrates and Tigris Rivers in Mesopotamia.

Mr. Frank S. Krug, Chief Engineer of the Department of Public Works of Cincinnati, Ohio, has reorganized the department as follows:

Mr. F. L. Raschig, M. Am. Soc. C. E., has been promoted from the position of Assistant Engineer in charge of the division of structures to be Principal Assistant Engineer in charge of the division of structures at a salary of $3600 per annum. Mr. Raschig is a graduate of the University of Cincinnati and has been in the employ of the city for several years. He is Vice-President of the Engineers' Club of Cincinnati.

Mr. H. F. Shipley, former City Engineer of Cincinnati during 1910 and 1911, and for the past two years in private practice as a Consulting Engineer, has been appointed Principal Assistant Engineer in charge of the division of highways at a salary of $3600 per annum.

Mr. H. S. Morse, M. Am. Soc. C. E., for the past two years Engineer in charge of the division of sewers and special division of sewerage investigations, has been appointed Principal Assistant Engineer in charge of the division of sewers at a salary of $3600 per annum. Mr. Morse is a graduate of the Massachusetts Institute of Technology and was formerly connected with the sewerage improvement work at Louisville, Ky., as Resident Engineer of construction.

Mr. H. L. Conway, recently Assistant Engineer with the County Engineer of Hamilton County, Ohio, has been appointed Principal Assistant Engineer in charge of the division of street and sewer repairs at a salary of $3600 per annum. Mr. Conway was formerly in the city employ as Engineer in charge of the department of track elevation and subways.

OBITUARY

Douglass J. Chase, former General Superintendent of the Atchison, Topeka & Santa Fé Ry., died Feb. 20, at his home in Macon, Ill., aged 78 years.

Arthur M. Richards, former Division Superintendent of the Chicago & Alton R.R., at Bloomington, Ill., died at his home in Chicago, Ill., Feb. 26, aged 76 years.

Charles A. Morris, Consulting Engineer of the Hayward Co., New York City, died Mar. 8, at Los Gatos, Calif. He was 62 years old and noted as an inventor.

John Sebastian, whose retirement as Vice-President of the Chicago, Rock Island & Pacific Ry., was announced about two months ago, died Mar. 1, at his home in Chicago, Ill. He was 63 years old.

Augustus Stanwood, a pioneer manufacturer of wood-pulp paper, died at his home in Brooklyn, N. Y., Mar. 5. He was born in Augusta, Maine, 75 years ago. In 1862 he built a factory at Gardiner, Maine, and began the manufacture of wood pulp for paper.

Col. James F. O'Shaughnessy, who at one time promoted a projected Nicaragua Canal Co., died Mar. 5, at his home in New York City. Following the Civil War he organized one of the first cotton-seed oil factories in the South. At the time the Nicaragua Canal was projected Col. O'Shaughnessy organized a company with a capital of $60,000,000 to undertake the construction.

Benjamin Illingworth, a pioneer steel maker and well known in the trade for many years, died in Jersey City, N. J., Feb. 22, at the age of 93 years. He was born in England and came to this country when he was 25 years old. He established and conducted steel works at Newark, Rockaway, Pompton and Jersey City. He retired from active participation in his various enterprises about 25 years ago.

Bradford H. Locke, President and Manager of the Locke Drill Co., New York City, died Feb. 22, at Lexington, Mass. He was a native of Massachusetts and graduated at the Massachusetts Institute of Technology in mining engineering in 1872. After extensive mining experience in California and Colorado he came to New York City, where for the last twelve years he had given his whole time to developing and marketing the Locke electric drill. He was 64 years old.

Richard D. Dodge, a retired Civil Engineer and formerly Instructor in engineering at the United States Naval Academy at Annapolis, Md., died Mar. 5, at the home of his son in Brooklyn, N. Y. Mr. Dodge graduated from Rensselaer Polytechnic Institute in the class of 1860—he was a classmate of the venerable William H. Searles, author of the well known engineers' pocketbook. He served in the engineering corps of the Navy during the Civil War, and was afterward Instructor at the Naval Academy. He was 75 years old and had been retired for some 20 years.

William C. Kirkland, Principal Assistant Engineer in charge of drainage, Sewerage and Water Board, New Orleans, La., died in that city Mar. 3, from apoplexy. Mr. Kirkland was born in Canada in 1861 and his early engineering experience was in railway work in Canada. He came to the United States in 1890 as a draftsman with the Charleston, Cincinnati & Chicago R.R., at Johnston, Tenn. In 1894 Mr. Kirkland went to New Orleans, where he was employed in the Topographical Survey Department of the City Engineer's Office. Two years later he was promoted to be Assistant Engineer of the Drainage Commission, later he was Chief Engineer of the Commission. Upon the consolidation of the Drainage Commission and the Sewerage and Water Board he was appointed Principal Assistant Engineer in charge of drainage. He was a widower and is survived by five children, four girls and a boy.

John C. Stuart, Assistant to the President of the Erie R.R., died Mar. 3, at Garden City, L. I., N. Y. From 1909 to October, 1913, he was Vice-President of the Erie R.R., resigning at that time because of ill health. Mr. Stuart entered the employ of the Chicago & Northwestern Ry. in 1880, and remained with this railway and the Chicago, St. Paul, Minneapolis & Omaha Ry. for 20 years, rising to the grade of General Superintendent of the latter railway in 1898. In 1900 he went to the Baltimore & Ohio R.R. as General Superintendent of the Middle and Northwestern divisions. The year following he was promoted to be General Superintendent of Transportation of the Baltimore & Ohio R.R. In 1903 he joined the operating staff of the Erie R.R. as General Superintendent at Cleveland, Ohio. He was promoted to

be General Manager in 1904 and elected Vice-President in 1909.

Dr. Edwin J. Houston, Past-President of the American Institute of Electrical Engineers, and widely known as the author of numerous textbooks, died, Mar. 1, at his home in Philadelphia, Penn., aged 67 years. He was born at Alexandria, Va., July 9, 1847. At 17 years of age he graduated at the Central High School at Philadelphia, where he subsequently became an instructor in natural philosophy. He was chiefly famous in the history of electrical engineering as one of the inventors of the Thomson-Houston system of arc lighting on which the Thomson-Houston Co. was founded, which was prominent in the early development of electrical engineering. Dr. Houston was Professor Emeritus at the Central High School, Professor Emeritus of Physics at the Franklin Institute and at the Medico-Chirurgical College in Philadelphia. He was a member of the United States Electrical Commission and Chief Electrician at the International Electrical Exhibition at Philadelphia in 1884. He was a member of the American Institute of Mining Engineers and twice President of the American Institute of Electrical Engineers besides being an active and contributing member of many other learned societies. He was the author of many text-books on electricity, physics, forestry, physical geography, and in recent years of several volumes of juvenile fiction.

Andrew Horatio Reeder, Vice-President and General Manager of the Stonega Coke & Coal Co., and allied coal, coke and railroad interests at Big Stone Gap, Virginia, died Feb. 26, at the Hahnemann Hospital in Philadelphia, Penn. Mr. Reeder was the eldest son of the late General Frank Reeder, of Easton, Penn., ex-secretary of Pennsylvania, and a grandson of Andrew H. Reeder, first Governor of Kansas during the border warfare days previous to 1860. Andrew H. Reeder was born Sept. 9, 1869, at Easton, Penn., and graduated with honors at Lafayette College in 1890. After a year's post-graduate course at Lafayette he became Chief Engineer for mining interests owned by the late Senator Elkins and ex-Senator Henry G. Davis, of West Virginia. Later he had charge of the Carnegie coal properties known as the H. C. Frick Coal & Coke Co., at Uniontown, Penn. For a number of years he was a consulting engineer and in that capacity he was employed by the Virginia Iron, Coal & Coke Co., to inspect and recommend improvements at its Toms Creek, Va., plant. Here he obtained such successful results that he was induced to remain in charge of the plant as General Superintendent. After 18 months at Toms Creek he went to British Columbia to become General Manager of James J. Hill's Crows Nest Pass Coal Co. After a period here he returned to Pennsylvania as General Manager of the Stonega Coke & Coal Co. Mr. Reeder was married in 1895 to Miss Elsie Longstreth Eckhard, of Philadelphia, who, with two children, survives him.

ENGINEERING SOCIETIES

COMING MEETINGS

ARKANSAS ENGINEERING SOCIETY.
Mar. 16-17. Annual meeting at Pine Bluff, Ark. Secy., P. B. Hill, Little Rock, Ark.

AMERICAN RAILWAY ENGINEERING ASSOCIATION.
Mar. 17-20. Annual convention at Chicago, Ill. Secy., E. H. Fritch, 900 S. Michigan Ave., Chicago.

HIGHWAY ENGINEERS' ASSOCIATION OF MISSOURI.
Mar. 18-20. Annual meeting at St. Joseph. Secy., Ray L. Cargill, St. Joseph, Mo.

EFFICIENCY SOCIETY.
Apr. 4-11. First annual conference in New York City. Offices at 41 Park Row, New York City.

AMERICAN ELECTROCHEMICAL SOCIETY.
Apr. 16-18. Spring meeting in New York City. Secy., Prof. J. W. Richards, Lehigh University, Bethlehem, Penn.

AMERICAN DRAINAGE CONGRESS.
Apr. 22-25. Annual meeting at Savannah, Ga. Secy., J. R. Gray, Chamber of Commerce, Savannah, Ga.

IOWA STREET AND INTERURBAN RAILWAY ASSOCIATION.
Apr. 22-25. Annual meeting at Waterloo, Iowa. Secy., H. E. Weeks, Tri-City Ry. & Light Cos., Davenport, Iowa.

Boston Society of Civil Engineers—The annual meeting will be held on Mar. 18 at the Boston City Club, Boston, Mass. It will comprise three main features: The business meeting when the officers for the ensuing year will be announced; the 32d annual dinner, at 2 p.m., at which Nelson P. Lewis, Chief Engineer of the Board of Estimate and Apportionment of the City of New York, will give an illustrated talk on "City Planning from the Standpoint of the Engineer," and the usual informal smoker, held in the auditorium at 7 p.m. The Society rooms are in the Tremont Temple, Boston.

American Water Works Association—The 34th annual convention will be held in Philadelphia, Penn., May 11-15, with headquarters at the Bellevue-Stratford. The Secretary is J. M. Diven, 47 State St., Troy, N. Y.

American Road Builders' Association—The Directors, on Feb. 24, decided to hold the 1914 convention in Chicago during December. It has been announced that earnest endeavor was made to arrange a joint convention with the other national good-roads organizations, such as the American Highway Association, American Automobile Association, etc., but that it was found impractical since the Highway Association had already arranged to hold its 1914 meeting in Atlanta, giving the Road Builders' Association no chance to coöperate in selecting a convention city.

Details of arrangements for the Chicago meeting and the accompanying good-roads show are in the hands of the Executive Committee, G. W. Tillson, E. L. Palmer, and R. A. Meeker. J. H. MacDonald, former Highway Commissioner of Connecticut, has been appointed a Director to fill the unexpired term of A. W. Dean, who was elected Second Vice-President at the last annual meeting.

National Drainage Congress—The fourth annual meeting will be held at Savannah, Ga., Apr. 22-25. Sir William Willcocks, of Cairo, Egypt, the English engineer who built the Assuan dam, and has been in charge of drainage and irrigation work in Egypt and Mesopotamia for many years, has accepted an invitation to address the congress on river regulation and control in ancient times. Preparation for the meeting is in the hands of a local committee, whose chairman is Mayor R. J. Davant, of Savannah; the secretary is J. R. Gray, of the Chamber of Commerce, Savannah, who should be addressed for further information.

National Fire Protection Association—The next annual meeting will be held in Chicago, May 5-7. The secretary is Franklin H. Wentworth, 87 Milk St., Boston, Mass.

Air Brake Association—The 21st annual convention will be held in the Convention Hall of the Hotel Pontchartrain, Detroit, Mich., May 5-8. The matters under discussion will include the following: (1) Electro-pneumatic signal system for passenger trains; (2) Air hose; (3) Clasp type of foundation brake gear for heavy passenger equipment cars; (4) Air gages and conductor's valves in caboose cars; (5) Analysis of factors involved in controlling and stopping passenger trains; (6) 100% efficiency of freight trains; (7) Recommended practice; (8) Mountain grade work; (9) Modern train building. W. J. Hatch is President and S. M. Mellis, of 53 State St., Boston, Mass., is Secretary.

American Society for Testing Materials—The 17th annual meeting will be held at Atlantic City, N. J., June 30-July 4, with headquarters at the Hotel Traymore.

American Railway Engineering Association—The 15th annual convention will be held in Chicago, Mar. 17-20, with headquarters at the Congress Hotel. On Mar. 17 and 18 various committee reports will be presented, and officers will be elected for 1914. The annual dinner will be held in the evening of Mar. 18.

American Peat Society—The eighth annual meeting will be held at Duluth, Minn., Aug. 20-22. Papers will be read on the uses of peat—agricultural, fuel, industrial, etc.

Philadelphia Association of Members of the American Society of Civil Engineers—A Philadelphia Association of members of the American Society of Civil Engineers was recently organized. The territory covered by the Association is Delaware and that part of Pennsylvania east of the Susquehanna, including the cities of Scranton and Wilkes-Barre. Meetings will be held regularly at the Engineers' Club of Philadelphia in January, April and October. The officers are: President, Geo. S. Webster; Vice-Presidents, R. L. Humphrey and F. Herbert Snow; Directors, J. S. Deans, J. W. Ledoux, Professor Edgar Marburg and H. S. Smith; Treasurer, S. M. Swaab; Secretary, W. L. Stevenson, 1122 Girard St., Philadelphia.

Master Boiler Makers' Association—The annual convention will be held at the Hotel Walton, Philadelphia, Penn., May 25-28. The Secretary is Harry D. Vought, 95 Liberty St., New York City.

Highway Engineers' Association of Missouri—The annual meeting will be held in St. Joseph, Mo., Mar. 18-20. Among the papers to be presented are "Hard Roads and Permanent Bridges," by Daniel B. Luten; and "Earth-Road Construction and Maintenance," by M. O. Eldridge. A banquet will be held on Mar. 19. The Secretary is Ray L. Cargill, St. Joseph, Mo.

International Railway Fuel Association—The sixth annual convention of the Association will be held at the Hotel La Salle, Chicago, Ill., May 18-21. The Secretary is C. G. Hall, C. & E. I. R.R., 922 McCormick Building, Chicago.